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Hydraulic Dredging on the Upper Mississippi River

By R. Monroe*

That part of the Mississippi River extending from St. Paul, Minn., to the mouth of the Missouri River is being improved for navigation by the United States to obtain a depth of 6 ft. at low water. To obtain this depth the low-water flow is being confined by closing side channels and building wing dams from shore into the stream to the proposed edge of the low-water channel. The width of this channel varies from 300 ft. at St. Paul to 1160 ft. at the mouth of the Missouri River. The bank-to-bank width of the river at St. Paul is about 1000 ft. and above the mouth of Missouri about one-half mile. From this it may be seen that the improved channel is quite narrow in proportion to the bank-to-bank width of the river. The improved channel, as a rule, will follow the concave banks, crossing from one bank to the other as the river winds along. It is upon these crossings that the shoalest water is found and here the hydraulic dredge has been found particularly effective in keeping an open channel for boats.

There is an immense amount of sand and silt brought into the Mississippi from side streams and especially from the Chippewa River, which empties into the Mississippi at the foot of Lake Pepin, 77 miles below St. Paul. This moving sand causes constant shifting of the steamboat channel and must usually be excavated by the hydraulic dredge to get it out of the channel.

While the important work of the hydraulic dredge at low water is to help maintain a navigable channel for boats and to deepen canals, harbors and landing places, a no less important work is building the foundations for wing dams when the river is at higher stages. A great saving in the cost of dam construction can often be made in this way. Sand can often be found close by that can be dredged by the hydraulic dredge and deposited in the base of a proposed dam at a cost of about 10c. per cu.yd. in the place of rock and brush costing on an average of $1 per cu.yd.

Three 18-in. dredges were completed last year by the United States and there are now in this district eight dredges as follows:

Name	Centrifugal sand pump size, in.	When built	Length of pipe line Suction	Length of pipe line Discharge	Dia. of pipe line, in. Suction	Dia. of pipe line, in. Discharge	First cost of dredge and pipe line
Geyser	10	1893	65	520	12	12	$5,450
Hecla	15	1901	65	500	18	15	27,380
Pelee	15	1909	65	600	18	17	36,130
Vesuvius	15	1909	65	600	18	17	33,375
Etna	18	1909	80	500	18	18	29,415
Mayon	18	1912	80	500	18	18	38,500
Taal	18	1912	80	500	18	18	38,500
Apo	18	1912	80	500	18	18	38,500

U. S. Hydraulic Dredge "Pelee" at Work on the Upper Mississippi 95 Miles below St. Paul, Excavating from the Steamboat Channel and Discharging on Shore

These dredges are essentially similar, the principal difference being the size and boiler and engine details. The principal features on each are the same, i.e., a centrifugal sand pump connected by rope drive to a steam engine, a swinging suction pipe supported by a catamaran and about 500 ft. of discharge pipe supported by pontons. Each plant consists of the dredge, suction ponton, 10 to 12 discharge pontons, 2 coal barges, 1 quarterboat, 1 motorboat and 2 or more rowboats. The services of a steamboat are required occasionally to move this plant from one dredging locality to another and to deliver coal to the dredge.

The principal equipment of each dredge consists of a battery of boilers, one main engine connected by rope drive to a centrifugal sand pump, auxiliary engines for swinging and hoisting the suction pipe, steam capstans and electric-light plant. All the dredges except the "Geyser" and "Hecla" have cross-compound condensing engines.

The following description of the "Apo" applies also to all the 18-in. dredges:

Hull—The hull is a composite structure of wood, steel and reinforced concrete, 130 ft. long, 28 ft. wide on deck and 5.4 ft. deep. The bottom rakes and gunwales are of creosoted Oregon fir. The framework is of steel angles and tees riveted together, consisting of six trusses running fore and aft with 4-ft. centers and 63 cross frames spaced approximately 2 ft., c. to c. The deck is 3-in. reinforced concrete, supported

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by the trusses and cross frames. The cabin is single deck, with one partition between the boiler and engine room.

Main Engine and Pump—This is one (Ball Engine Co.) cross-compound (12 in.–25x16 in.) condensing engine connected by rope drive to one (Morris) 18-in. centrifugal sand pump.

Boilers—There are three (horizontal cylindrical) of the Mississippi River type in one battery, each boiler having 5-in. flues. Each boiler shell has a diameter 40½ in. and length of 20 ft. 2½ in. The battery of boilers has a grate surface of 48.75 sq.ft. and heating surface of 1166 sq.ft. The furnace is equipped with the Hulson shaking grates and two 12-in. Wing turbine blowers.

Donkey Boiler—One vertical cylindrical fire-pot boiler 36x16 ft., used when washing main boilers.

Feed-Water Heater—There is one closed heater of the steamboat type, 8 in. in diameter and 34 ft. long, containing 6 lengths of 4-in. iron pipe.

Feed-Water Pumps—These are two of the Worthington ram pattern duplex pumps, each 5¼x3½x5 in. One of these also furnishes water to cool the main pump shaft bearings when necessary.

Condenser—The condensing apparatus consists of one Deane single horizontal direct-acting vacuum pump with jet condenser 10x16x18 in.

Priming Pump—This consists of one 6-in. centrifugal (Morris Machine Co.) pump direct connected to a vertical steam engine 5x5 in. This pump is used to prime the main sand pump.

Hoisting Engine—This consists of one American Hoist & Derrick Co., single-drum hoisting engine with two cylinders 5½x8 in. each and is used for hoisting and lowering the suction pipe.

Swinging Engine—This consists of a two-cylinder 6x8-in. reversible engine with two drums and is used for swinging the suction ponton. It is popularly known as the "Clinton Midship Nigger."

Steam Capstans (2)—These are the No. 5 Hyde Windlass Co., capstan with two 4½x6-in. cylinders. One is located near each end of the dredge and they are used with the holding lines in moving the dredge during operations.

Spud Hoists (2)—These are simple steam cylinders, one at each end of the dredge and used for hoisting the spuds. The size of each cylinder is 12 in. x 6 ft., and they set upright along the side of and are directly connected by the piston rod and automatic friction clamp to the spud.

Electric-Light Plant—This consists of one General Electric 110-v., 4.5-kw., direct-connected multi-polar generator driven by a vertical, single-cylinder, 4½x4-in. center crank engine.

Dredging Operations

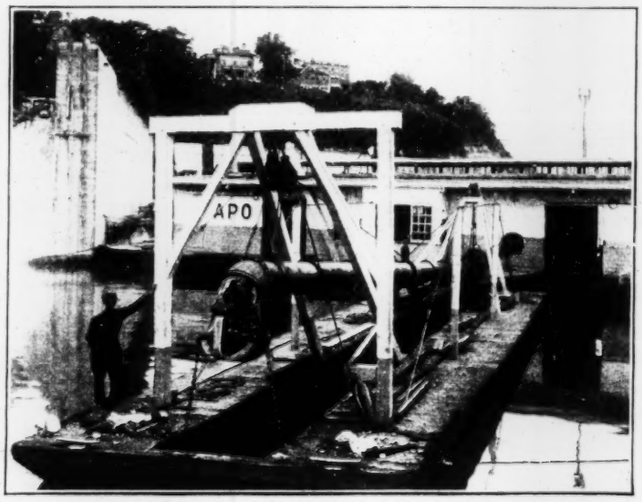
The dredge usually has a double crew with an Inspector or Surveyor in local charge, all subsisted by the United States and living aboard the quarterboat. Operations are begun at 2 a.m. and continue to 10 p.m. each day excepting Sundays and holidays during the season of navigation. The crews change watch at 6 a.m., noon and 6 p.m., each working a total of 10 hours per day. Those on the early morning watch are off duty between 6 a.m. and noon and are on duty again between noon and 6 p.m. The crew on watch from 6 a.m. to noon go on again from 6 to 10 p.m. The authorized double crew consists of 18 men as follows:

1 Surveyor, in local charge	$75.00 to $125.00	per month
1 Receiver of materials of timekeeper	50.00 to 60.00	per month
2 Enginemen (one on each watch operating dredge and pipe line)	80.00 to 110.00	per month
2 Enginemen (one on each watch operating auxiliary engines)	65.00 to 75.00	per month
2 Firemen (one on each watch)	45.00	per month
6 Dredge hands (3 on each watch acting as coal passer, spud tender and oiler respectively)	35.00	per month
1 Watchman	40.00	per month
1 Boatman (oarsman or motor skiffman)	40.00	per month
1 Cook	60.00	per month
1 Waiter	25.00	per month

18 Total.

Before dredging is begun at any locality, it is customary to make a survey extending well over the area to be dredged and for a short distance above and below it. From the map of this survey the area to be dredged is determined and the approximate quantities are obtained. Another survey is made after dredging and from these two surveys the conditions before and after dredging are shown and means provided to check approximately the calculated quantities of dredging. Soundings are taken from the dredge both forward and aft as the dredging proceeds, the length and width of cut measured and from these dimensions the quantities of dredging credited to the dredge are obtained.

In dredging in the open river, operations are begun either at the upstream or downstream end of the area to be dredged. These dredges having end discharge, i.e., where the discharge pipe leaves the end of the dredge, may begin either above or below, while the side discharge dredge begins below the area to be dredged. In either case the dredge is put in position broadside to the current and held by spuds and holding lines of 1½-in. manila rope. The anchors to which the holding lines are attached are placed before dredging is begun. These anchors are usually the (so called) disk anchor and are jetted down into the river bottom 7 to 8 ft. They are a cone-shaped disk about 16 in. in diameter with a small



Ponton and Suction Head of U. S. Dredge "Apo" at Keokuk Lock in Des Moines Rapids Canal

opening in the center through which the jet acts in sinking and raising the anchor. These anchors are sometimes placed by the steamboat and crew and at other times by the dredge and crew. The apparatus necessary is a small force pump with hose and a length of 2-in. pipe. The outer end of the pipe is reduced to allow it to pass through the center opening in the disk anchor a few inches. When two anchors are placed and lines are run from them to each end of the dredge it is then necessary to place another anchor to which a line from the discharge pontons may have a fastening. Where the dredging is close to shore it is often possible to get fastenings there for one or two holding lines. After this the dredge is hauled by the lines into the desired position and dredging operations may be begun.

The center or side line of the proposed dredging is usually marked by range flags on shore or buoys in the water, or both, and during the dredging operation the dredge is kept upon these marks.

Soundings are taken to help locate the point to begin dredging and the suction pipe is lowered to the depth required and held there by its own weight and chains con-

necting the suction head to the floating catamaran directly above it. The inner end of this catamaran has a pin connection to an A-frame hinged to the side of the dredge about amidship. The suction pipe has an accordion flexible joint at this pin connection, thus allowing the catamaran to sweep through an arc of 180°. The outer end of the catamaran is connected by cables running to each end of the dredge through sheaves to a swinging engine located about amidship of the dredge. The catamaran is swung back and forth by this swinging engine when the dredge is in operation and as the catamaran is about half the length of the dredge it is possible, with each sweep, to travel about the semicircle whose diameter is the length of the dredge. In dredging in shoal places this provides a cut sufficiently wide to allow the dredge to pass through broadside. It is not customary in practice, however, to swing the full semicircle because of the small angle between the swinging cable and the fore and after center line of the catamaran at extreme positions of the latter. The circle swept through is nearer 130° and the dredges make cuts in this way of 80 to 100 ft. in width.

Each time a sweep of the suction catamaran is made, the dredge is moved forward into the cut approximately 2 ft., varying somewhat according to the depth of dredging. In this way first one end of the dredge and then the other is moved into the cut. At each move the spud at the end of dredge to be moved is raised and the holding line slackened or tightened two feet and the spud dropped again. If the dredge is working downstream, the holding lines are simply slackened after the spud is raised and the dredge drops down with the current, but in working upstream the holding lines are placed on the steam capstan and the dredge pulled upstream about two feet each move. In this way the dredge walks up or downstream with the suction sweeping back and forth constantly and removing all loose material above the desired grade.

The material after passing through the pump is discharged ashore or as far away from the cut as the length of the discharge pipe will allow. To remove sand and mud in this way from the river bottom a large percentage of water is raised with the material. A sample taken from the pump showing 25% solids might be considered a very good average. A velocity of 10 to 15 ft. per second is necessary to keep the coarse materials moving through the discharge pipe.

It is very seldom possible to run the dredge continuously during the 20-hour working day. Aside from the time lost in moving from one locality to another, moving out of the way of passing boats and moving the ponton line, there are many little delays due to clearing obstructions from the suction pipe, shutdowns for repairs, etc. But with all these interruptions the hydraulic dredge moves material at a very low cost compared with the dipper dredge or other excavating machines. In dredging to improve the channel it is customary to make a cut approximately 200 ft. wide. This width is necessary for boats to navigate and to meet and pass safely in the open river.

During the latter part of the season of 1912 eight dredges were in operation and the dredges worked at 62 localities. The following tabulation given at the top of the next column shows the work for the season for each of the eight dredges.

Name of dredge	Dates in operation	Dredging cu.yd.	Total cost*	Cost per cu.yd.
Geyser	7/19/12 to 11/7/12	67,922	$7,880.84	$0.116
Hecla	5/31/12 to 10/30/12	190,120	9,850.19	0.051
Pelee	5/4/12 to 10/31/12	303,365	18,601.23	0.061
Vesuvius	4/24/12 to 11/7/12	427,956	15,026.48	0.0304
Etna	5/2/12 to 10/20/12	422,024	17,514.48	0.041
Mayon	9/5/12 to 11/14/12	158,737	6,031.05	0.038
Taal	7/15/12 to 10/31/12	202,221	7,384.14	0.0365
Apo	8/8/12 to 10/30/12	73,123	9,225.82	0.126
Total		1,845,468	$91,514.23	$0.0495

*Cost does not include any charge for care, repair or use of plant.

The records show 1,845,168 cu.yd. of sand, mud, gravel, etc., dredged during the calendar year of 1912, and a total of 8848 hours actual dredging or pumping time for the eight dredges. This shows an average of 208.58 cu.yd. per pumping hour per dredge, costing approximately $0.0495 per cu.yd., not including plant rental or extraordinary plant repairs.

The "Etna" made the best record for any one day during the season. This was at Mile 229, below Guttenberg on Aug. 6. An area of 48,400 sq.ft. having an average depth of 6.18 ft. was covered and 11,078 cu.yd. of material removed in 18 hours and 13 minutes, or an average of 608 cu.yd. per hour. This is the best record so far as known for any dredge boat on the Upper Mississippi River.

To properly lay out the area to be dredged through a bar is sometimes a problem. To show the situation clearly it is often sufficient to plot the survey and draw in contours at low water and 6 ft. below low water, on a scale of about 400 ft. to 1 inch. Upon this drawing the proposed dredging can be outlined satisfactorily if the line of least resistance through the bar follows the direction of the current. If the current does not follow this direction, it is then a more difficult matter to locate the dredging properly. To dredge across the current is evidently a waste of effort, as such a cut soon fills up with moving sand. Another difficulty arises here because of the fact that the direction of the current changes at different stages of the river. At high water the current is more directly downstream and will not wind across from bank to bank as it does in low water. This tendency of the current to change at different stages makes it difficult to locate the dredging to satisfy all stages. This applies more generally to the wide flat places in the river. Inasmuch as it is most important to keep the dredge cuts open at low water it is evident that the cuts should lie in the direction of the low-water current.

This means that at high water the dredged channel may be partially or entirely obliterated. For this reason dredging of this character is not permanent, except to the extent that a certain amount of material has been placed back from the channel between the wing dams where it stays and gives no further trouble. It may also aid by helping to contract the width of the waterway. It is claimed that dredging and removing the material from between the shore lines will lower the water surface or reduce the velocity of the flow, but this is not noticeable in hydraulic dredging on the Mississippi River, as the cross-section is usually increased very slightly in proportion to the size of the river.

There is an immense amount of sand to be removed to obtain and maintain a channel with 6 ft. at low water in the Upper Mississippi River. Even after the works of contraction have been completed and the desired depth obtained, the dredge will still be a necessity to remove quickly from the channel deposits brought down by high water and side streams.

Colorado Street Bridge over the Arroyo Seco, Pasadena, Calif.

On the outskirts of Pasadena, Calif., there is now being built, across the Arroyo Seco, a valley 1500 ft. wide and 175 ft. deep, a reinforced-concrete bridge with arches of especially light form for so large spans. It has many points of interest, particularly low cost, a curved plan and a peculiar system of centering. It is being built jointly by the city of Pasadena and the county of Los Angeles, both of which will be served by its roadway.

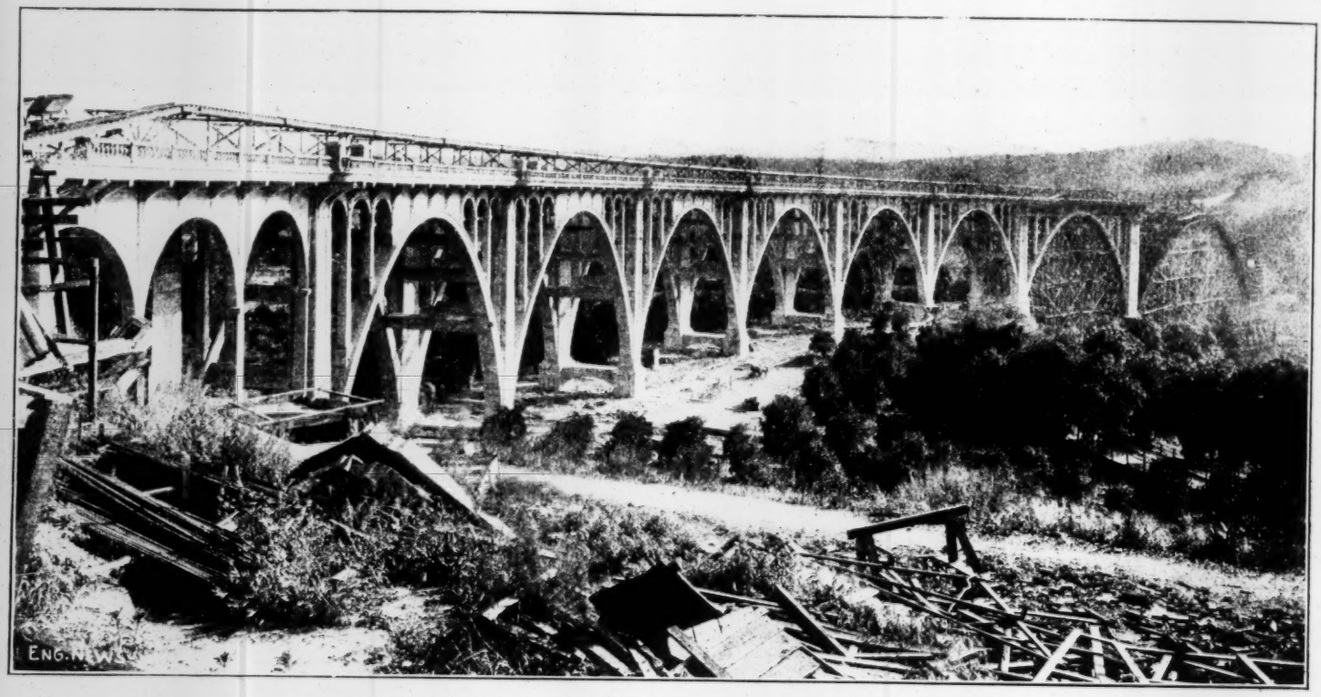


FIG. 1. THE ARROYO SECO BRIDGE AT PASADENA, CALIF.
(Looking west toward the 210-ft. arch, in course of construction in extreme background.)

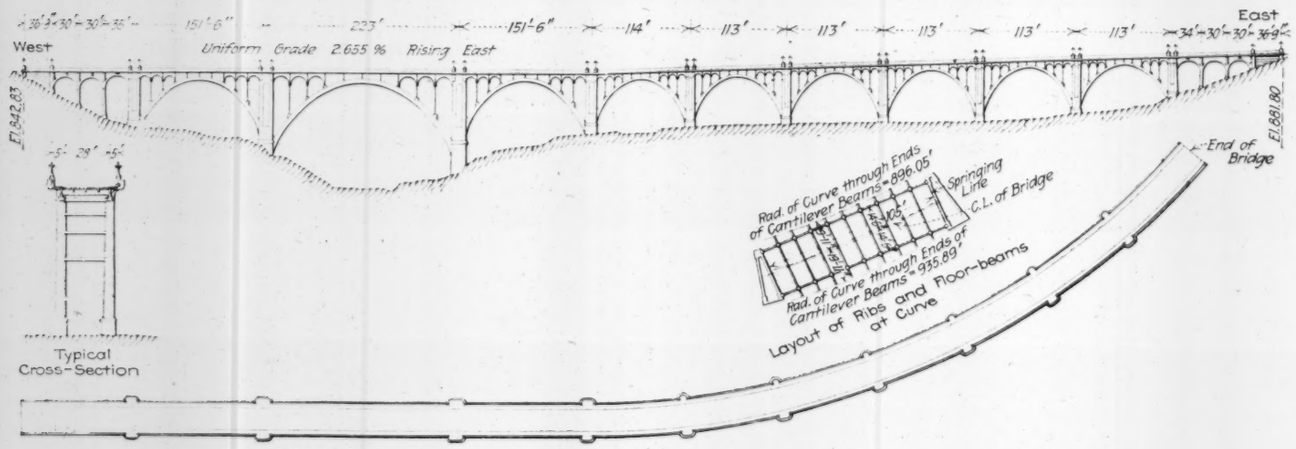


FIG. 2. PLAN AND ELEVATION OF COLORADO ST. BRIDGE, PASADENA

Fig. 2 gives the plan and elevation of the bridge. As will be noted there, it is about 1470 ft. long, of which about 800 ft. is on a curve and the remainder on a tangent. It comprises, going from the east, a 37-ft. retaining wall, three 30-ft. arches, five 113- and one 114 ft. arch spans, all on the curve; a 223-ft. and a 151-ft. arch span on the tangent, and three short arches with a retaining wall, completing the structure. All the spans given are from center to center of piers. The curved alignment was used because it afforded a better location for the pier than would have a straight alignment. As will be noted in Fig. 2 the roadway follows the curve but the arches are on the chord, an arrangement allowed by designing the piers on the curve in triangular form, as shown. The bridge has a 28-ft. roadway and two 5-ft. sidewalks, with recessed bays at each pier. It is designed with a rising grade toward the east of slightly over 2.6% . It is to be used only for highway and pedestrian traffic, no allowance being made for railway tracks of any kind.

DESIGN

As noted above, the lightness of the design is a distinguishing characteristic. This can be observed in the de-

ails of the arch spans in Figs. 3 and 4. All of the arch spans are of the same general type, varying only in size. They are made up of two parabolic arch ribs nearly square in section, carrying spandrel columns, which in turn carry both tranverse and longitudinal beams. The reinforcement and design of the arch structure and the details of the transverse and longitudinal floor beams are shown also on the figure.

Fig. 4 gives the details of the smaller arches, which

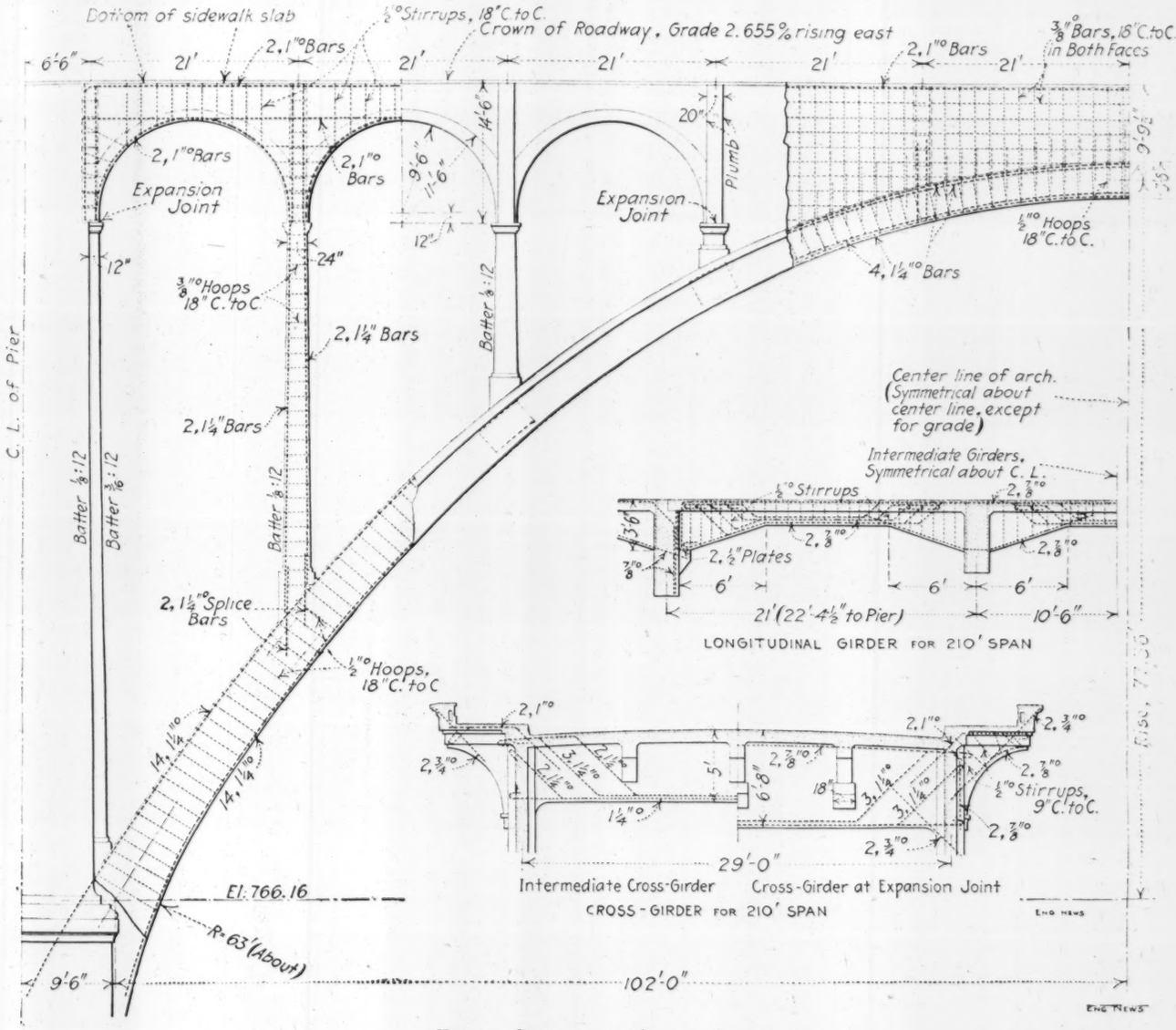


FIG. 3. DETAILS OF LARGE ARCH

[210 ft. c. to c. of ribs at skewback.]

DIMENSIONS OF ARCH RIB

	Depth in.	Width in.
Spring	76	78⅝
Col. 1	64⅛	72
Col. 2	54⅜	66⅜
Col. 3	48⅜	62¾
Crown	44	60

reinforced-concrete slabs frame into these beams. In addition to the bracing of these floor-slab cross-beams, the arch ribs are further braced by tie beams between the two ribs at each spandrel column.

Fig. 3 gives the details of the large arch, which is 210 ft. center to center of the ribs at the springing line and about 204 ft. between faces of the piers. In this arch the ribs are spaced 29 ft. center to center. They are 44 in. deep at the crown and 60 in. wide, increasing uniformly to a depth of 76 in. and a width of 78⅝ in. at the springing line. The short table under Fig. 4 gives the dimensions at various places on the arch rib.

Fig. 4 gives the details of the smaller arches, which are miniature replicas of the larger spans. They have a crown depth of 2 ft. and a crown width of 3 ft. 8 in., increasing to a depth of 3 ft. 6 in. and a width of 4 ft. 8½ in. at the springing line. Expansion in all of the arches is provided for by sliding joints at the base of the spandrel arches and by vertical joints at the beginning of the spandrel wall, all shown on Figs. 3 and 4.

A typical pier on the tangent is shown in detail in Fig. 5. The piers, it will be noted, rest on the solid rock which is very near the surface all the way across the valley. They comprise two shafts rising independently

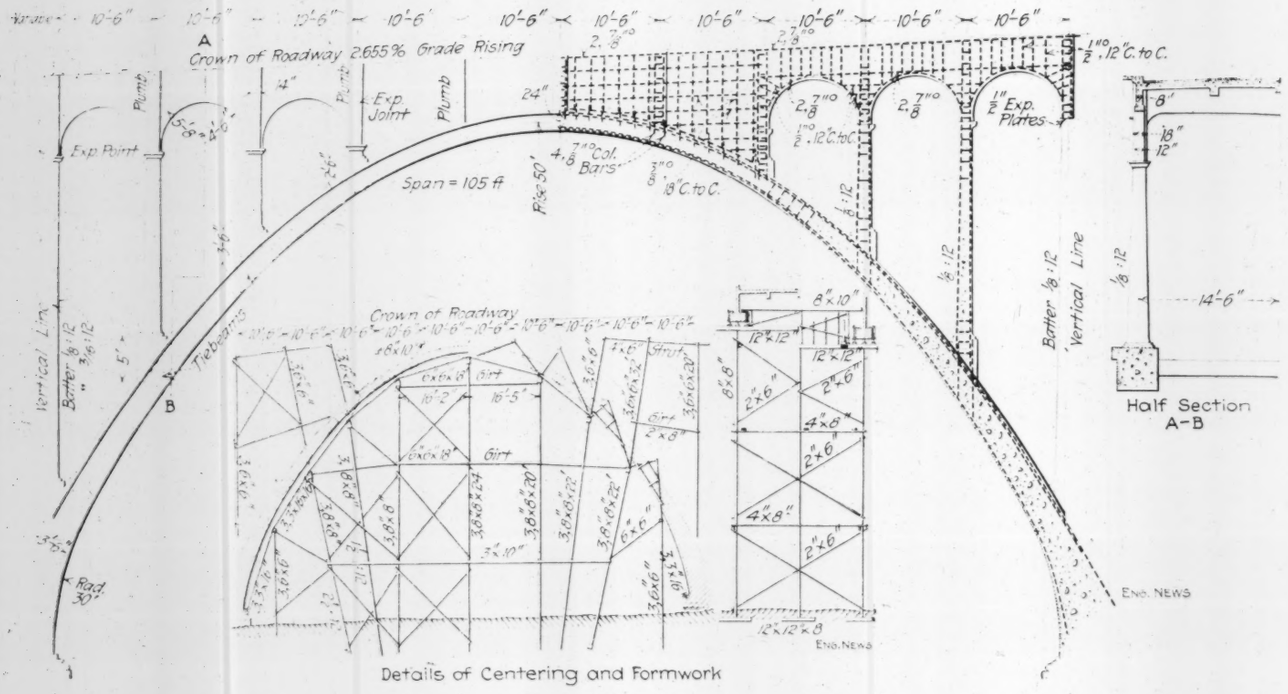


FIG. 1. DETAILS OF 105-FT. ARCH WITH CENTERING

to the elevation of the floor system and braced transversely by a small arch just below the springing line. On the curved section of the bridge, the design of the piers is somewhat different, in that the inside of the curve has a single shaft in plan and the outside a double shaft, each connecting arch being in duplicate, and parallel to the floor beams on its respective side. This is shown well in the detail in Fig. 2.

LOADS AND STRESSES—The engineers have given out the following outline of the loads and stresses used in designing the structures:

The structure is designed for a live load consisting of class B of Waddell's "De Pontibus," with impact, determined by the formula $I = \frac{100}{L + 150}$, which consists of a load varying from 123 lb. per sq.ft. for a loaded length of 100 ft., up to a maximum of 166 lb. per sq.ft. The following road-roller loads were considered: an ordinary 15-ton road roller with which permissible stresses of 500 lb. per sq.in. of compression on the concrete and of 15,000 lb. per sq.in. tension on the steel were considered; and an emergency road roller weighing 21 tons for which loading the above intensities were increased to 650 lb. and 20,000 lb. per sq.in. respectively. The wind loads figured were 30 lb. per sq.ft. of exposed surface for the unloaded structure, and 15 lb. per sq.ft. of exposed surface for the loaded structure. Except as mentioned, the allowable unit stresses are: For concrete in compression, 600 lb. per sq.in.; for concrete in tension, zero; for concrete in shear when combined with tension: (a) when without reinforcement, 35 lb. per sq.in.; (b) when a portion of the tension reinforcement is bent 50 lb. per sq.in.; (c) where the web is reinforced to carry diagonal tension, 120 lb. per sq.in.: For steel in tension, 15,000 lb. per sq.in. When considering wind and temperature changes the above stresses are increased 30%. These working stresses were

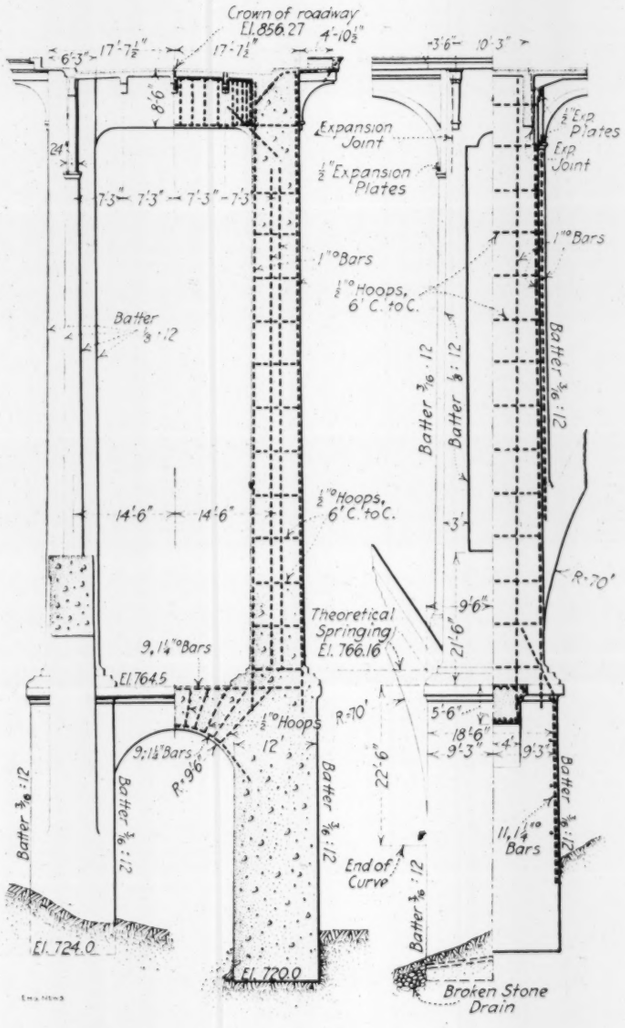


FIG. 5. DETAILS OF PIER 9 BETWEEN 210-FT. AND 140-FT. SPANS

used with certain other limitations in design. In the calculations every possible condition of partial loading of single spans and of adjacent spans was considered, and the extreme conditions provided for. As before stated, most of the piers are founded on boulders and gravel and a maximum intensity of 5½ tons per sq.ft was allowed on the foundation.

Construction

It is thought that the construction of the bridge, which is the particular province of the contractor, is being carried out in as economical a manner as was the design, for which the engineer is alone responsible. This economy is due mostly to the fact that an over-equipment

Fig. 6. View at One of the Piers under Construction

of plan has been avoided, only just enough plant being provided to carry on the work with the proper assistance of manual labor. This may be noted principally in the method of mixing and placing the concrete.

The concrete mixing plant consists of elevated bins for the sand and for the gravel, discharging by gravity into a chute where the aggregate is measured and the cement brought in from an adjacent storage house, also by gravity. A gasoline-driven concrete mixer takes the material and discharges it mixed into dump cars which run on a track carried on a trestle supported on the centering or later on the partly finished roadway, following the center line of the structure. This track, which may be noted in Fig. 1, rises until it is about 12 ft. above the finished roadway, and then continues on the uniform 2.65% grade of the bridge throughout the length. The trains of cars carrying the concrete buckets are controlled by a cable which hauls the cars up a short grade and over the hump and then controls them while they run by gravity to their proper location. After dumping, they are drawn back by the engine with sufficient speed to carry them over the summit when run by gravity down to the mixer.

The concrete is dumped from the cars into a movable distribution hopper, whence it is dumped through a controlled opening into steel troughs, from which it slides to its final position in the form. Only two men are required to handle the concrete mixer, one man at the hoisting engine and one at the receiving hopper. From four to six men place and spade the material in the forms, making a gang of from 8 to 10 men, who sometimes place as high as 120 cu.yd. of concrete in one working day.

The materials for the concrete are taken to their respective bins by motor trucks, while the timber for the centering and forms is hoisted to place by a team of horses. All other material requiring lifting is moved by a team or by manual labor, and no other power is used in the construction.

Falsework and Forms—Fig. 4 shows the details of the falsework centering for one of the smaller arches. Each of these arches is the same in design, so that the same work could be used over and over again. The design for the larger arches is practically the same, differing only in size.

The centering for the smaller arches consists of 8x8-in. posts with 2x6-in. bracing; three posts per bent are used, one under the center of the roadway and one under each arch rib. The caps are 12x12-in. timbers which extend clear across. As will be noted, only the central bents are made vertical, the others being leaned toward the piers. The sheathing and side forms for the ribs are supported on three joints under each rib, which rests on the wedges at each cap. The forms for the tie beams between arch ribs are carried by small trusses of 2x6-in. pieces spaced about 3 ft. 10 in. center to center and resting on the two adjacent caps. Fig. 1 will show the appearance of the centering in the completed arch.

In general, planed-timber forms were used for all concrete work, though for some of the special work, such as the cantilever beams, the capitals on the spandrel columns and other small projections, prepared sheet-metal forms were employed.

Material—There are about 11,000 cu.yd. of concrete and 600 tons of reinforcement used in the bridge. The specifications provide for the concrete to be as follows, for the greater part of the bridge: One part cement, 2 parts sand, 4 parts stone or gravel, to pass a 1-in. ring. The piers below the coping to the springing line are of 1:3:5 stone or gravel to pass a 2½-in. ring; boulders are imbedded in this portion where they will not interfere with the reinforcement rods. Shafts of piers above springing line are 1:3½:7 concrete in small aggregate. Reinforcing is of corrugated bars rolled from openhearth steel of ultimate strength 60,000 to 70,000 lb., with a minimum elastic limit of 35,000 lb.

No special treatment of surface finish is being provided on account of lack of funds, but the forms are made tight enough so that the finish will be sufficiently pleasing to allow its use without further treatment; if

such treatment is desired in the future it can be done if funds are provided.

Personnel

The construction is being carried on under the direction of Waddell and Harrington, consulting engineers, Kansas City, Mo., with C. K. Allen as the local representative. The contract is held by the Mercereau Bridge & Construction Co., Los Angeles, Calif., under the special direction of F. W. Crocker of that firm, who is responsible for most of the construction methods.

The lump-sum contract for the bridge, complete, ready for traffic, is $188,000, to which there may be added some amount for additional depth of piers. As the valley has a width of nearly 1500 ft. and a depth as great as 175 ft., this is not a high cost for a bridge which, from Fig. 1, will be seen to be decidedly monumental in character.

A Channel- and Beach-Maintenance Problem

By R. R. Raymond*

SYNOPSIS—Absecon Inlet is on the north shore of Atlantic City, N. J., and the problem of maintaining a navigable channel across the bar at the entrance is complicated by the probable effect of jetties upon the famous Atlantic City beach. The factors to be taken into account in improving tidal estuaries are interestingly and comprehensively stated, and the effects of proposed solutions of the Absecon Inlet problem are carefully explained. Maj. Raymond has decided that the best method of maintaining the channel without injury to the beach is by dredging only, using for that purpose a specially designed dredge. The work will be experimental and will be watched with much interest by harbor engineers.

The improvement of an inlet on a sandy coast involves a number of factors which are not all apparent until the problem is thoroughly studied, and which are not easily controlled even when understood.

Tidal Currents

In order to approach intelligently the improvement which is the subject of this brief and incomplete article, let us examine some of the principal factors which we must control so as to produce a navigable channel depth.

Tidal Range—Without going into the theory of the ocean tidal waves, let us start with the fact that on the New Jersey coast the surface of the ocean rises and falls twice each day. At Absecon Inlet this fluctuation, commonly called the tidal range, is about 4 ft.

Supposing that there exists within the coast line a landlocked basin connected with the ocean by an inlet, we can readily conceive that when the rising tide raises the ocean surface above the level of the water in the basin, the ocean water will pour through the inlet, and the basin will tend to fill up. Likewise, we can understand that when the ocean level falls below the basin level the water will run out again.

If the ocean were to remain stationary at its high-tide level until the basin were filled to the same level, the amount of water that would flow in through the inlet would be the full tidal capacity of the basin. But this full tidal capacity is seldom, if ever, admitted in nature, for the ocean tides do not wait for the basin to fill or empty.

To make this clear, let us suppose that the inlet is deep enough to act at low tide but is very narrow. The water will pour in at a rate dependent on the head, and the total amount that can enter during the time that the ocean level is above the basin level may be so small that the basin level will be very little affected. When the ocean tide is low, the basin will discharge, and again the amount that will pass will be too small to lower the basin level to that of the ocean. Thus the water in the basin will never rise as high at high tide nor fall as low at low tide as the water in the ocean.

The narrower the inlet, the greater will be the tendency for the basin level to remain at the height of half-tide in the ocean; and the wider the inlet, the more closely will the surface levels of the basin and the ocean agree.

Tidal Volume—By multiplying the area of the basin by its average tidal range, we compute the tidal volume that will flow in and out through the inlet; and by dividing this volume by the average cross-section of the inlet, we determine the average velocity of flow through the inlet.

It is a very common error to assume that the velocity of flow can be increased in proportion by contracting the cross-section of the inlet. The fallacy lies in assuming that the tidal volume will not be affected by such contraction, whereas we have just seen that a smaller inlet can pass only a smaller volume. To contract the inlet simply shuts out the flood tide and reduces the tidal volume. It is a very different matter from contracting the channel of a river, whose discharge remains the same.

Varying Cross-Section of Inlet—Another important point, frequently overlooked, is that the surface of the water at the inlet rises and falls with the surface of the ocean, while the surface of the water in the basin is always lower than the ocean level when the tide is running in and higher when the tide is running out. But the area of the cross-section of the inlet varies according to the depth of the water thereat.

Considering any level of the basin, we see that at that particular stage during flood tide the cross-section of the inlet is larger than it is at the same stage during ebb tide. This is true for all stages; the average cross-section of the inlet during flood tide must be greater than the average cross-section during ebb. Yet the total amount of water that flows in equals the amount that flows out. For this reason, the velocity of the ebb flow through the inlet is greater than the velocity of the flood tide.

Other Factors Affecting Tidal Currents at Inlet

Again, the tidal capacity of a basin is very largely dependent on the depth and form of the basin as well as on its area.

Depth of Basin—Considering depth first, suppose a long watering trough to be empty and a bucket of water

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to be poured in at one end. The water flows along the bottom of the trough until a thin film of water lies over the full length. But if the trough be nearly full when the water is poured in, there is no great lateral flow induced. The increased depth is affected by a slight movement throughout and the added water remains at the end where it was poured in.

In the same way, if the basin be shallow the tidal volume must flow laterally to all parts; but if the basin be deep there will be less lateral movement of its waters.

In a deep basin the water admitted by the inlet is quickly absorbed, while in a shallow basin it is so retarded near the inlet that the basin level at this place may rise so nearly with the ocean level as to greatly reduce the slope of the water in the inlet and so cause a reduced flow.

Thus if two basins have the same form and area and equal inlets, and are subject to the same ocean tidal range, the tidal volume will be larger in the deeper basin.

Indian River Inlet, Delaware, is an excellent example. This inlet connects the ocean to Indian River Bay and Rehoboth Bay, whose combined areas should give, apparently, a large tidal volume and maintain a large inlet. But these bays are so shallow that the effect of the tide admitted is lost in the immediate vicinity of the inlet. The greater part of the area has no tidal fluctuation. The inlet is almost closed.

Another example is Chincoteague Bay in Virginia and Maryland. Here there is a large inlet with good access to the ocean and bay; but the bay is too shallow to give a free flow to the tides and the greater part of the bay feels no effect from the strong tides at the inlet.

Barnegat Bay, New Jersey, is another good example having no tidal fluctuation except near the inlet; while Pensacola Bay, Florida, is an illustration of the opposite kind. There the bay is large and *deep,* and the tides flow in freely, giving a very large tidal volume, although the tidal range in the Gulf of Mexico is not half as great as that on the New Jersey coast.

Form of Basin—The influence of form also is important. If the basin be somewhat circular, less resistance to the flow of the tides will be found, and a shorter lateral movement of the water be required, than will be the case if the basin be irregular and straggling with numerous long and tortuous branches.

Summary—To summarize: the tidal flow in an inlet depends on the tidal range in the ocean, the area, form and depth of the basin, and the area and form of the cross-section of the inlet. There are other factors, such as the length of the inlet channel and the direction and force of the wind; but ordinarily those enumerated are the controlling factors, although at times the wind may dominate.

Sand Movements

Having outlined the action of the water, we may turn to a consideration of the sand. Without touching upon the source of supply, we may accept the known fact that there is a progressive movement of sand toward the south along the New Jersey coast at and near Absecon Inlet.

This sand is very fine indeed. A handful of it does not seem gritty but rather soft and smooth. If a layer of it be placed at the bottom of a tumbler of water and the water be gently set to rotating by stirring at its upper surface, the sand will readily rise and cloud the water, settling again when the water is allowed to come to rest.

Every wave that breaks on the shore moves sand up the beach and back again, generally bringing it seaward a little farther south. In this way the sand moves to our inlet, and the waves and the flood tide rushing in carry the sand along. But when the current expands inside the basin, its velocity is checked and the sand settles to the bottom.

And now becomes apparent the vital importance of the fact, pointed out above, that the ebb is swifter than the flood; for the ebb is able to pick up the sand from where the flood drops it and sweep it out and deposit it in the ocean so far out that the feebler flood current cannot bring the same sand back again, but must draw its supply from the littoral drift from the north. The sand swept out by the ebb forms a bar outside of the inlet. This is always the case with a tidal inlet on a sandy coast.

If our basin were to receive the discharge from one or more rivers of sufficient size, the superiority of the ebb tide over the flood would be greater, perhaps so great as to keep a good channel across the bar. Unfortunately, Absecon Inlet has no large river to help it.

Examining the map on the following page (Fig. 1) we see that the inlet is nearly a mile wide, but that the sand brought in from the north forms a great shoal on the north side extending about three-quarters of the way across. Over this shoal the waves and the floor currents continually bring in sand. This is swept out by the stronger ebb current.

Since the supply is all on the north side, the ebb succeeds best in scouring along the south side of the inlet, and there we find deep water. There would be a tendency for the inlet to move bodily southward, due to the accretion of sand on the north and the erosion on the south side, were it not for the protection in the form of bulkheads, spur jetties, etc., placed on the south side to hold the shore line of Atlantic City in a fixed position.

The sand blown out through the deep channel is dropped when the ebb current disperses into the ocean. Here is formed the outer bar which is the great obstacle to navigation. Not only is the depth insufficient for vessels of a desirable size, but the channel shifts about, always seeking to avoid the pile of sand which it is always itself building in new places.

Proposed Improvements

The objects sought in improving this inlet are to provide a suitable depth for navigation and to keep the channel fixed in position. Incidentally it must be observed that no plan should be adopted which would interfere with the sand movement now supplying the beach of Atlantic City south of the inlet. On this beach the sand is always progressing toward the south. If the supply from the north be interrupted, there can be no doubt that the beach will disappear and Atlantic City will have to choose between a sea-wall and destruction. Sand catches can do no good unless there is sand to catch. Even a sea-wall might not stand if the sand should wash from beneath it. Some sand would probably still be driven ashore by the ocean, but the cutting off of the present supply could not do otherwise than change the existing balance and produce great changes in the beach. Thus any plan providing for the construction of a jetty must be approached with caution.

The danger to the beach might be avoided by placing a jetty at such a position as not to check the littoral drift of sand; but no jetty, however safe as regards its action on the beach, should be built without a definite understanding of its probable effect on the navigable depth.

Effect of a South Shore Jetty—Looking at the map we see that there is a good depth along the inlet shore on the south side. This has been explained above. If a jetty were built to extend this shore line outward into the ocean, say along the broken line *MN*, the deep channel would follow out along the jetty, pushing the sand discharged with the ebb farther out into deeper water. The immediate result would be a deeper channel; but gradually sand would accumulate beyond the jetty and a new bar would be formed. The remedy would be to extend the jetty oceanward or to dredge away the sand.

Either remedy would be temporary in its effects, for the bar would build again. If a jetty were used, the new bar would be farther out and the cost of the jetty would increase very rapidly as it reached deeper and deeper water, while its growing length and exposure would increase the cost of maintenance.

Such a jetty would probably have a marked effect on the beach and its adoption would be unwise. Considering its effect on the navigable depth, it would be more effective than a jetty on the north side of the inlet, as explained below, but it would not produce a permanent channel across the bar and would never justify its own cost.

Effect of a Jetty to Narrow the Inlet—Probably the most popular treatment of the inlet would place a jetty on the line *AB* for the purpose of concentrating the ebb flow into the deep channel. This plan would lose its popularity as soon as put into execution.

Fig. 2 is an ideal cross-section of the inlet looking toward the ocean. Let us consider the effect of a jetty built out from Brigantine across the flats to the edge of the deep channel at the point marked *B*. Since this jetty would not project out into the ocean, it would not interfere greatly with the littoral movement of sand and probably would not do any serious damage to the Atlantic City beach.

Let the line *CD* (Fig. 2) represent the level of the water in the inside basin at, say, half-tide. During flood tide the ocean level will be higher, say at the line *AB*; while during ebb tide the ocean level will be lower, say at the line *EF*.

Now if we construct a jetty as stated from Brigantine out to *B*, we shall reduce the area of the flood-tide channel by the whole area from the line *AB* to the bottom, while the area of the ebb-tide channel will be reduced only by the area from the line *EF* to the bottom.

The area *AEGH* is the difference between the present flood and ebb channel areas. No survey has measured

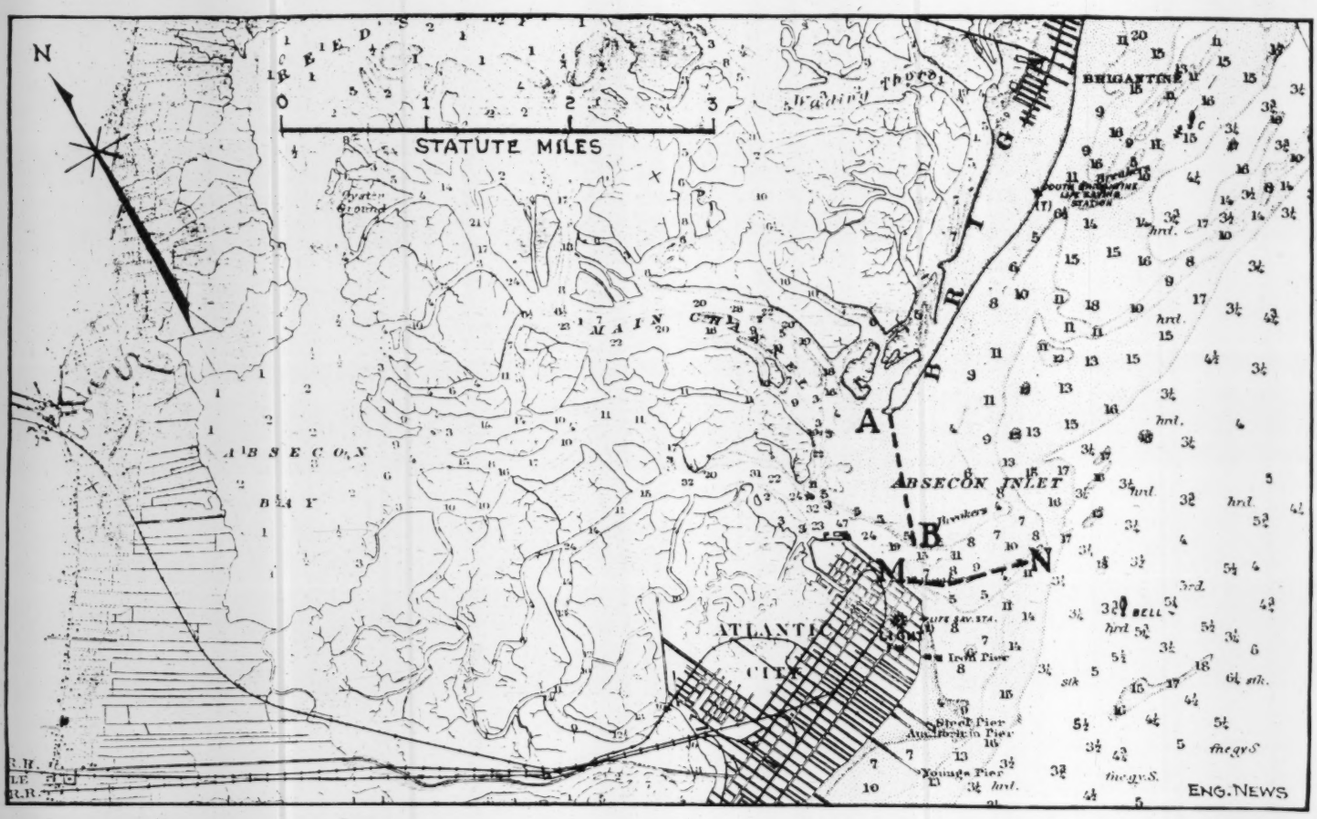


Fig. 1. Map of Absecon Inlet Showing Proposed Jetties; from a United States Coast and Geodetic Survey Chart

(Figures in shaded portion are soundings in feet; on white, in fathoms.)

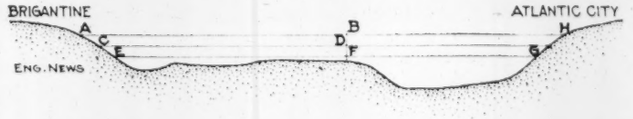


Fig. 2. Cross-Section of Absecon Inlet

it in recent years, but the writer's personal observations lead him to believe that it is probably not less than 4500 sq.ft. at half-tide.

The effect of the present conditions is to allow the flood tide the whole cross-section of the inlet up to the line *AH*, filling the interior basin very much fuller than could possibly be the case if the only entrance were restricted by a jetty to the channel from *B* to *H*.

While we are considering just now only the half-tide stage, it must be observed that the same principles apply to any stage assumed except the high- and low-water stands, when the flow ceases in order to reverse its direction.

The same amount of water that comes in during flood tide flows out during ebb. The cross-section of the ebb channel is now limited, without a jetty, to the area below *EG*, and the velocity of the ebb in the channel is greatly increased over that of the flood by this natural contraction.

Under existing conditions, then, the inlet shoal between Brigantine and *B* acts like an automatic valve, freely admitting the flood tide over the shoal, and then forcing the greater part of the ebb discharge out through the channel.

To close this valve entirely by building a jetty would be to throttle the flood current, reduce the tidal volume of the interior basin, and decrease the power of the ebb. A violent current would be produced at the gorge of the inlet, but its effect would be largely local and the channel over the outer bar, where the real obstruction to navigation is found, would be shoaled.

Theoretically it should be possible to build a jetty whose top could be placed at such a level that the flood tide would pour over it to fill the basin while the ebb would be cut off. Thus a jetty built up to the line *EF* would give the flood tide an entrance whose area would be *AEFB* in addition to the channel area up to the line BH, but it would allow the ebb tide no outlet except the channel area up to the line *FG*.

Practically, although at the first blush the scheme seems to promise success, the action of such a submerged jetty is apt to be more or less uncertain; and it is doubtful whether its result would be any improvement over the present excellent natural valve constructed and maintained in a wonderfully delicate adjustment by natural forces.

Effect of Other Proposed Jetties—Other locations for one or two jetties are available and have received considerable study, but any jetty outside of the inlet must affect the littoral sand movement, and any jetty inside must interfere with the flood current to a greater extent than with the ebb.

The history of an inlet provided with jetties usually exhibits a very large expenditure for jetties, followed by additional expenditure to alter them in an attempt to make them effective, and finally a resort to dredging. The latter is almost invariably successful, and generally at an annual cost less than a reasonable interest on the cost of the jetties. Where dredging is tried first, jetties rarely, if ever, are later adopted except for protection of the channel.

Maintaining a Channel by Dredging Only—At Pensacola Harbor, Florida, jetties were once proposed. They would have cost about $2,000,000 if they had been built. The entrance is now maintained at the desired depth of 30 ft. by dredging alone. The cost of the dredge was $172,221 and the dredging for seven years prior to July, 1912, cost $100,701, of which only $268 was for maintenance. At 4% the interest on the cost of the jetties for seven years would have been $560,000, and it is probable, if not certain, that auxiliary dredging would have been required.

Dredging at Pensacola, however, is a much simpler matter than dredging at Absecon Inlet; for at Pensacola there existed a natural depth of 19.5 ft. due to the large deep interior basin, while at Absecon Inlet the present depth is in some places only 4 ft. within the limits of the desired channel. This depth requires that the earlier stages of the work be carried on at high tide in order that the dredge may float; and it is apparent that such a condition adds to the danger of the dredge's grounding and being left there by the falling tide. The danger is peculiarly great at Absecon Inlet on account of the fineness of the sand and its rapid erosion when subjected to eddies.

The immediate result of the grounding of a vessel on this bar is the formation of eddies around and under the hull, due to the ever-active ground swell. The sand beneath the hull is whirled away and deposited generally as soon as the eddy currents lose themselves in the quieter water near by. There is formed about the vessel a basin surrounded by an encircling shoal, and the removal of the vessel becomes every minute more and more difficult.

When the tide falls, the scouring away of the sand brings the hull into contact with the compact sand below, which is almost unyielding as granite; and the incessant motion of the ground swell, raising and dropping the vessel, soon starts the plate and frame rivets.

Such conditions can be met only by a vessel having so light a draft as to minimize the probability of grounding and possessing efficient means of decreasing her draft at a moment's notice and applying great power to draw her from the toils of the sand. A thorough canvass of the Atlantic and Gulf coasts was made in 1912, but it failed to discover a single dredge competent to work at a reasonable cost at Absecon Inlet.

Having rejected jetties as a means of effecting an improvement, and having selected dredging as a possible solution of the problem offering good chance of success, we are confronted by the question how to dredge under the very difficult conditions on the bar. These conditions are now at their worst, for after a channel of convenient size has once been made, the great danger of grounding the dredge on the bar, there to be destroyed by the heavy ground swell, will exist only at the edges of the channel, and by dredging the edges during the rising tide and at other times working in the safer places, this danger can be reduced to a minimum.

One objection that has been advanced against the probability of successful dredging on the bar is the large number of days when it will be unsafe to work. It should not be assumed, however, that the dredge will be idle necessarily when not able to work on the bar.

The intimate relation existing between the tidal flow through an inlet and the depth and form of the interior basin and its branches has been pointed out elsewhere in this article. It is probable that dredging to improve the tidal flow in the various thoroughfares opening into the interior basin may produce a great and beneficial effect

on the bar, possibly as useful as the dredging of an equal amount from the bar itself.

This proposition has not been thoroughly investigated as yet for Absecon Inlet, and indeed its possibilities have not generally been appreciated. The writer cannot recall at present any reference to it which lays proper emphasis upon its importance. Nevertheless, he has found it to be a fact that the improvement of an interior channel may produce a remarkable result at points nearer the ocean.

A recent illustration is furnished by the improvement of Leipsic River, Delaware, a tidal stream flowing into Delaware Bay. Four miles or so below the headwaters of this river and about 13 miles above its mouth the channel has been restricted for many years by a county bridge whose approaches were earth embankments. The only opening through this virtual dam was 36 ft. wide., and the tidal currents just at the bridge were swift and strong.

At some little distance below the bridge there existed a shoal or bar, and in preparing for dredging about to begin in the upper river this bar was staked out as a place requiring work. But before dredging this shoal the dredge was taken to the old bridge, the abutments and approaches were dug out and the tide was given an unobstructed channel to the upper river.

Then it was discovered that the shoal down the river had disappeared before the dredge could go to work upon it. The removal of the obstruction has so improved the tidal flow that the effect is felt all the way to the mouth and across the bar in the bay.

Special attention is invited to the fact that although the contraction of the channel by the bridge abutments and approaches had caused locally a large increase in the velocity of the tidal flow, nevertheless, it had reduced the tidal volume and the scouring power of the currents throughout the river. A similar result should naturally be expected if Absecon Inlet should be contracted.

Dredge of Special Design—Close study of Absecon Inlet would probably discourage the average dredging expert. The great difficulties and dangers would be apparent to him and his acquaintance with ordinary types of dredges would convince him that they could not be used there successfully. In this he would be right. Ordinary dredges were never designed and constructed to meet such conditions.

In all the earlier investigations of Absecon Inlet by the Corps of Engineers, dredging was considered as an auxiliary to jetties, and careful consideration of jetties developed their great cost and uncertain action. The inlet was given up in despair.

A later report proposed dredging alone, but placed the estimate of cost at a prohibitive figure. A study of this report indicates that the real elements of the problem were not appreciated and that little or no consideration was given to the possible development of a dredging plant able to meet the actual conditions. This is not said in criticism, for the problem was outside of ordinary engineering practice. A dredge was proposed but no indication was given that it should be in any respect different from dredges for other ocean bars. The difficulties were seen to be great; therefore the cost was estimated to be large. There was nothing in the proposed dredging to promise success and nothing was done to put it into execution.

Atlantic City was not willing that the improvement should be abandoned without at least an attempt at its consummation, and requested a further investigation which led to the belief that dredging might be successful if wisely directed so as to coöperate with the natural forces. This, it was foreseen, would be possible only with a dredge of special design.

A board of engineer officers was appointed which reported in effect that the promise of success was sufficient to justify the construction of a dredge to try the experiment. Congress has now supplied the funds to carry out the project proposed by the board.

Since this last report was submitted, the inlet has been the subject of further study, with the result that additional special features for the dredge have been found necessary. The plans are now being worked out in the office of the Chief of Engineers and present several novel departures from former designs. It is thought that this dredge deserves a description at greater length than would be permissible in the present article, which pretends only to outline the principal points of interest in the proposed improvement.

It should be remembered that this dredge is designed to attempt a difficult feat under new and untried conditions. Even if unsuccessful, its cost will not be wasted, for it can be assigned to work at some less difficult inlet. But the writer believes that, properly used, it will prove a success.

Method of Dredging

The purpose of the plan is not to make a deep cut across the bar in the usual way, but to continually disturb the sand at the crest of the bar and always at the point where the natural forces tend to place the channel, incidentally removing a considerable quantity of sand in the dredge hoppers in addition. Since it has been found that the operation of a few steamers over the bar, always on the same line, maintained a deep but narrow channel, it seems reasonable to expect the dredge, with its actual ploughing of the sand and its positive removal of considerable quantities, to maintain a much better and wider channel.

Once a groove is established in the natural line of flow of the ebb, the tendency will be for the ebb to maintain it. Sand will encroach from the north, it is true, but such sand, freshly deposited, will be readily disturbed and eroded as well as dredged. The compacted sand of the bar is very hard and firm, making the channel easier to hold in a fixed position.

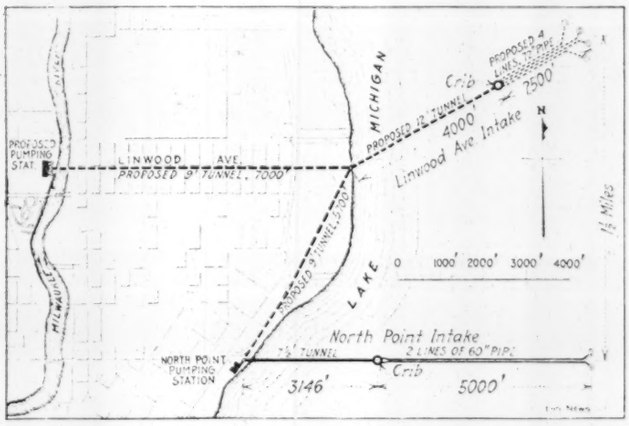
The greatest part of the dredging is to be expected along the northern edge of the channel, where the littoral sand comes in. Storms from the northeast may cause a large fill in the channel from time to time, but such storms are usually accompanied by storm tides, flooding the marshes and driving in through the inlet so vast a tidal volume that the succeeding ebb flow is extraordinarily powerful and helps to restore the channel at once. The deposit from a storm is not compact and is readily eroded or dredged. Doubtless there will be times when the channel will be obstructed, but such periods should be brief as well as infrequent.

It must be acknowledged that the attempt to dredge will be experimental until its success or failure shall be established by actual results. It is well worth a trial, however, and that trial is soon to be made.

The Proposed Water Works Intake Tunnel at Milwaukee, Wis.

The construction of a new water-works intake tunnel has been commenced by the city of Milwaukee. It will consist of 4000 ft. of brick tunnel, 12 ft. in diameter, extending to a crib out in the lake in water 40 ft. deep.

From the crib it is proposed to extend four lines of 6-ft. cast-iron water mains to a point 2500 ft. out in the lake where the water is 60 ft. deep. This plan is shown in the figure.



PLAN OF PRESENT AND PROPOSED INTAKE TUNNELS FOR THE MILWAUKEE WATER SUPPLY

A contract was entered into with Joseph Hanreddy, of Milwaukee, on April 29, for the construction of the first section, consisting of the shore shaft, the lake shaft, and 4000 ft. of brick tunnel, at the following contract prices: shore shaft, $300 per ft.; lake shaft, $500 per ft.; 12-ft. brick tunnel, $87 per ft.; and for any rock excavation, $12 per cu.yd.

It is estimated that two years will be required to complete this contract covering the first section of the intake tunnel.

It is proposed to later build two land tunnels from the shore end of the new 12-ft. intake tunnel, each having a diameter of 9 ft. The first will extend along Linwood Ave., and just across the Milwaukee River, a distance of 7000 ft., where it will terminate in a pumping station. The other tunnel will extend in a straight line along the lake front to the present North Point pumping station, a distance of 5700 ft.

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Encouragement of Forestation by the Canadian Pacific Ry.—At the recent meeting of the Canadian Forestry Convention, held in Winnipeg, Man., July 9, the rapid deforestation and the importance of some systematic scheme of reforestation of the great Canadian timber reserves of the Northwest were discussed by many notable men. Among the speakers was George J. Bury, Vice-President of the Canadian Pacific Ry., who is reported in the Montreal "Star" as follows:

The Canadian Pacific Ry., excepting the government of Germany, has probably made a more elaborate study of the preservation of forests than any organization in the world. The study of the natural resources of the country has been made by this company with a twofold purpose, to estimate the amount of traffic that may be derived from year to year in the territory adjacent to the line and what production can be made from timber, and still preserve it to such an extent that 100 years hence forestry preservation and timber production will be so advanced that the yield and the revenue will be greater in the same territory than today. It has been estimated that the revenue derived by a railway from the production of one acre of heavily timbered land is equal to the accumulated traffic of an acre of agricultural land for 80 years.

Asphaltic Concrete as a Paving Material for Residence Streets*

BY LINN WHITE†

The ideal pavement for all conditions has not yet been produced and probably never will be. On light-traffic streets, or those of mixed service, permanency is hardly so imperative as on heavily traveled streets, and the consideration of cost becomes more important. Here, also, the future growth or probable change from a district partly occupied by residences to one devoted entirely to business should receive careful consideration.

It is proposed to discuss here residence streets, boulevards and those serving suburban districts. The general character of the districts served may be considered fairly well established, so no great changes in kind of traffic may be anticipated. The traffic on such streets is seldom what is probably termed "heavy," but may be "intense," that is, there may be a great many rapidly moving, comparatively light vehicles. Automobiles will predominate, with a lesser number of horse-drawn pleasure vehicles. At times, heavy teaming of building material, produce, coal, etc., must be provided for, and the general delivery and express service for the district.

To satisfy the requirements of such cases, a pavement must have many high qualities, and the standard is all the time growing higher, the public exacting and expecting better service. It must be reasonably permanent, moderate and flexible in cost, serviceable on all kinds of foundations, sanitary, smooth, dustless, noiseless, non-slippery, resilient, and agreeable to the eye in strong sunlight—quite a formidable list of virtues difficult to attain.

Examining the different kinds of pavements we will at once pass up many of them as possessing in an eminent degree only a few of the qualities named. Brick, concrete, stone block and wood block all possess some of these qualities but are lacking in others. Macadam cannot be classed as a permanent pavement, and we are thus reduced to some form of bituminous paving surface.

Sheet asphalt (sand asphalt) pavement is the best known and most widely used form of bituminous pavement—at least, it has been so for years past, though now other forms are becoming extensively known. Sheet asphalt must, however, be made comparatively thick, say 3 in., to conform to accepted standards; its cost is comparatively high, dependent on its thickness, though not in direct ratio thereto; it is suitable for use only on a concrete base; has a smooth, sheet-like surface more or less slippery; and last but not least has fallen somewhat into disrepute because of the skill and care required to produce a successful paving mixture and the constant effort during recent years to reduce its cost.

Asphalt block is a pavement that has been in existence for many years, but has not come into general use because of its high cost and difficulty of manufacture and transportation.

Macadam surfaces bonded by the application of bitumen in a liquid form have come largely into use during recent years, but have not established themselves as a standard type of pavement because lacking in the very essential quality of permanency under considerable traffic. Their place is on the country highway and roads of occasional traffic.

Having thus briefly reviewed the principal varieties of street pavement known and used in this country, we come to the one remaining general variety, designated as "bituminous concrete." Under this general name is included all paving mixtures consisting of broken stone combined with sand or other fine mineral matter, cemented together by a bituminous binder, which may be either tar or asphalt, making either "tar concrete" or "asphaltic concrete."

While both tar and asphalt are hydrocarbon compounds, their qualities are quite different. Tar, for practical use in a pavement, cannot have a melting point much higher than 130° F., as it would become very hard and brittle in cold weather. In this climate it is not unusual to find temperatures of 115° or even 120° on the street surface, under which conditions a tar binder melting at less than 130° would be almost liquid and have but little cementing or bonding value. In other words, tar, while quite adhesive and ductile, is very susceptible to changes of temperature. For these reasons it will not be considered further in this discussion.

The various asphalts that may be used in an asphaltic

*Abstract of a paper read before the Civil Engineers' Society of St. Paul, Minn., Apr. 7, 1913. (Published in full in the "Journal of the Association of Engineering Societies" for June, 1913.

†Chief Engineer, South Park Commissioners, Chicago, Ill.

concrete have very different qualities and require different treatment, which will be briefly alluded to later.

For a paving mixture to merit the name of asphaltic concrete, the broken stone, according to a definition adopted by the American Society of Municipal Engineers, must be in sufficient quantity to form an important part of the mixture; also the ingredients must be combined and mixed before being laid. Neither the size of the stone nor the proportion of the various sizes in the mixture can be limited by the name "asphaltic concrete," any more than the size and proportions in a hydraulic concrete can be limited by the name. This can be done only by the specification of the particular mixture it is desired to produce.

Several varieties of asphaltic concrete are offered on the market under trade names, copyrighted or patented, that presumably conceal or but half reveal their somewhat mysterious qualities, such as "Bitulithic," "Amiesite," "Filbertine," "Warrenite," "Westrumite," etc. They are all asphaltic concretes founded on the old concrete idea of a broken stone aggregate supported and held together by a mortar matrix.

The first one named, "Bitulithic," is supported by letters patent in which the principle of filling the voids in the aggregate by the use of successively smaller sizes down to impalpable powder is exploited. It is claimed by this method of grading the sizes of the mineral that an "inherent stability" is produced in the pavement independent of the asphaltic binder, which then may be very soft and serve only to waterproof the mixture and to fill the remaining small percentage of voids.

Several patents have been issued to the originators of this pavement, the claims of which to the lay mind seem very similar, all of which rely upon this main idea of carefully grading the aggregate, and some of which describe carefully the process of separating the mineral into a number of sizes and recombining them according to a definite formula. This brings out the idea of "predetermining the sizes" of the mineral aggregate, which is a phrase met with in "Bitulithic" literature.

There is another claim in the "Bithulithic" patents that is an excellent one if carried out to its logical conclusion, that is, that by reason of filling voids so completely by grading the mineral the quantity of asphaltic binder may be reduced and the pavement "produced at a smaller cost." This is a quotation from the claims of the patent, and if produced at a smaller cost it naturally should be sold at a smaller price, thus justifying protection under the patent laws and earning the gratitude of the nation.

These references are made to the claims of the "Bitulithic" pavement, disregarding the other special asphaltic concrete pavements named above because the "Bitulithic" is the only one that endeavors so to broaden its claims as to monopolize the field of asphaltic concrete paving. If we sum up the claims of the "Bitulithic" patents, we find they hinge upon, and in fact are reiterations of, the principle of reducing voids by carefully graded sizes so that an "inherent stability" is produced and less bitumen used, thus reducing cost and increasing usefulness.

If it is true that "Bitulithic" methods are the best and cheapest, it becomes the duty of every engineer and city official to support them and use them. Let us examine the situation a little further and see what conclusions can be drawn on this point.

It cannot be contended that good "Bitulithic" pavements are not produced, for many excellent examples may be found throughout the country. At the same time many excellent examples may be found of sheet asphalt pavements, and it cannot be contended there is any "inherent stability" in the sand of which they are composed. All the stability must be supplied by the asphaltic cement.

In 1907 a contract was let to the Bitulithic Co. for about three miles of pavement on Michigan Ave. and South Park Ave. in Chicago, which was laid during 1907 and 1908. On these avenues there is but little heavy teaming, as they are boulevards and under usual boulevard restrictions as to traffic. In 1908 a traffic census showed from 3000 to 5000 vehicles per 24 hours on the three miles of pavement in question, which by 1912 had increased to from 5000 to 12,000 per 24 hours—practically doubling the average number.

During the summer of 1909 some small portions of the surface began to show a tendency to form waves and ruts, but was scarcely noticed, being considered only local or accidental defects. It was not thought there could be anything very serious the matter; the spell of "inherent stability" was too strong.

By the summer of 1910 conditions were much worse, and the contractors put heavy rollers on to smooth out the waves and ruts, also attempting to roll into the surface, first limestone screenings, then granite chips, and, finally, crushed granite about ¾ in. in size.

These expedients one by one failed. The screenings, chips and crushed granite could not be forced very deeply into the surface and most of the particles were easily loosened and torn out again by the traffic. It was found that while the rollers smoothed out the surface it wouldn't stay smooth, but quickly formed ruts again as long as warm weather continued. By keeping the rollers going more or less all the time until cooler weather arrived, the streets were kept in fair condition for the winter, but early in the summer of 1911 the campaign had to be begun again.

This season it was prosecuted more vigorously. The crushed granite was coated with bitumen to make it bond more effectually into the body of the pavement.

The surface of the street was punctured full of small holes into which the coated stone was swept and then rolled. These efforts were all of doubtful and temporary value. Conditions all the time grew worse, and it is safe to estimate fully $20,000, or from 25 to 30c. per sq.yd., was expended by the contractors during the three seasons named without materially improving the condition of the pavement.

In the summer of 1912, under strong pressure from the Park Board, the contractors abandoned their efforts to stiffen up the pavement and resurfaced the whole area with a layer of new asphaltic concrete about 1½ in. thick. The method followed was to go over it with surface heaters, softening up the surface until about an inch in thickness could be removed with rakes. Then the new 1½ in. layer of asphaltic concrete was spread and rolled, thus making it about ½ in. thicker than formerly. After this resurfacing the pavement was left in excellent condition at the end of the season of 1912 and so remains up to the present time.

It is to be noted the original pavement was specified to be at least 2 in. thick and actually averaged 2½ in. throughout a large portion of it; also that the original binder was tar. Thus the pavement is now composed of 1 to 1½ in. of tar concrete, overlaid with 1½ in. of asphaltic concrete.

In extenuation of the troubles on this particular pavement it may be pointed out that there are many excellent examples of "Bitulithic" pavement to be found. This is true. There is one other piece of "Bitulithic" pavement in Chicago, about one-half mile long, on Sheridan Road, which is as fine a piece of pavement as can be found, and it is older than the Michigan Ave. pavement. But it is from one failure like this that we can learn more than from numerous successes that teach nothing.

It may also be argued that the traffic on Michigan Avenue was extremely severe, or that if the original binder had been asphalt instead of tar the result would have been better.

This may be true, but it is not to the point. The main point is the very important "inherent stability" was not in the pavement to resist displacement independent of the support of the binder; and if it was there at all it was not sufficiently "inherent" to resist the severe traffic.

The traffic, while severe, was not prohibitive—was not sufficient to strain a good asphaltic concrete pavement beyond a safe limit. It was merely a severe test and developed a weakness that might have developed to a lesser degree or more slowly under lighter traffic.

In 1909 an asphaltic concrete pavement was laid on Michigan Ave., between Twelfth St. and Jackson St., which in 1911 was carrying a traffic of 17,000 vehicles during 24 hours. This was not laid by the Bitulithic Co. nor under their specifications, and was the subject of a suit between the Bitulithic Co. and the South Park Commissioners, of which more will be said later.

No trouble has occurred with this pavement up to date except a comparatively small number of cracks which have in every case been traced to the concrete base. In explanation it should be said the street between these points is 75 and 85 ft. wide between curbs, having been widened 35 ft. to the eastward when paved in 1909. Thus a portion of the sub-foundation is new filled ground and a portion the old street, and the support is uneven. Also the construction of a number of large buildings on the west side of the street has caused settlement in places and consequent cracks in the pavement. This pavement is between the "Bitulithic" pavement and the center of the city and carries a larger traffic.

There were also asphaltic concrete pavements laid adjoining the "Bitulithic" on Michigan Ave. at 39th St., and on South Park Ave. at 35th St. They are on both avenues extensions of the "Bitulithic" to the southward and carry the same traffic in slightly decreasing numbers. No trouble has occurred with these pavements and they have cost nothing for repairs up to date except on two intersections where the cross traffic was quite severe and has occasioned minor repairs.

As stated above, neither "Bitulithic" methods nor formulas were followed. No special care was taken to grade the aggregate except to be sure there was plenty of mortar to

fill voids and the fine material in the mortar was in proper proportions to give a well-filled mixture. Beyond this the success of the pavement depended on the proper preparation of the base and the use of a suitable asphaltic binder. Dependence was had on only common-sense methods to resist lateral displacement, and the paving mixture was intended to be waterproof for obvious reasons.

The heating and mixing were done in portable plants working on the street; the proportioning of the mineral ingredients was by wheelbarrow loads; and the prepared material was delivered quickly and directly on the street surface. Much of the success and moderate cost were doubtless due to the methods and machinery used. There was no overheating or chilling of material due to long hauls, no separation of ingredients from the same cause, and no bad work on account of delays either at the plant or on the street. The concentration of all operations in one place, under one superintendent, undoubtedly tended to better results in many ways.

The comparison of cost between the three sections of pavement referred to above is as follows:

The "Bitulithic" by contract cost $1.90 per sq.yd., exclusive of cost of base.

Asphaltic concrete by contract on Michigan Ave., between 12th and Jackson Sts., $1.73 per sq.yd., including concrete base, which would make it exclusive of base about $1.10 per sq.yd.

Asphaltic concrete laid by day labor on Michigan Ave., south of 39th St., and on South Park Ave., south of 35th St., less than 75c. per sq.yd., exclusive of base.

Following are the analyses of the mortar constituent (that is, 10-mesh material or less) of several samples of asphaltic concrete which have been under traffic for long enough periods of time to demonstrate their good or bad qualities. The first four proved to be very good, and the latter two indifferent in quality.

	No. 1 Jackson and La Salle	No. 2 Jackson and Main	No. 3 Michigan and Park	No. 4 Grand and Okwd.	No. 5 Jackson and Franklin	No. 6 Jackson and Market
	%	%	%	%	%	%
Bitumen	18.8	18.9	16.1	21.0	18.9	16.3
200-mesh	10.8	11.6	11.7	10.9	4.3	3.5
80-mesh	10.6	11.5	7.0	26.2	5.9	5.7
40-mesh	36.8	35.3	22.7	19.3	50.3	54.1
10-mesh	23.0	22.7	42.5	22.6	20.6	20.4
	100.0	100.0	100.0	100.0	100.0	100.0

As far as the coarse aggregate is concerned, that is, all material coarser than 10-mesh, there were considerable variations in the different samples, but the total quantity was approximately the same, and the variations seemed to have nothing to do with the success or failure of the sample. Samples 1 to 4 were successful, and they were rich in dust and reasonably well proportioned in other sizes up to 10-mesh. Samples 5 and 6 were practically failures, and they are seen to be lean in dust with an excess of 40-mesh material not well filled by the smaller sizes.

Here is where stability is found, or such "inherent stability" as belongs to the mineral aggregate of asphaltic concrete, not in the careful grading of the larger sizes. It would not matter if there was 25 or 50% of mineral larger than that passing a 10-mesh screen added to the mortars of samples 1 to 5; the result would have been the same. Nor would it matter if this 25 or 50% of coarse mineral was all of one exact size or of several carefully graded sizes; the pavement would have been good.

Therefore, in the making of good asphaltic concrete, it is an axiom that we must have a good mortar, well proportioned as to mineral, with a good asphaltic cement to bond it properly together. Having this, the coarse aggregate may be added with considerable freedom.

There are, of course, certain precautions that must be taken and certain useful refinements that may be attained, as in the making of other bituminous pavements. For instance, too much dust in a mixture will require more asphalt, etc. All such points could not be covered in such a discussion as this, but the principles stated above may be taken as fundamental.

There are on the market two general classes of asphalt binders, or asphaltic cements, as they are more properly called when of a character suitable for use in paving mixtures, distinguished generally from each other by the presence or absence of any considerable degrees of ductility.

Ductility is the quality that enables a substance to be drawn out like chewing gum without breaking. It is of no direct value in the paving industry, but is supposed to indicate other qualities of value, such as cohesion and adhesion. At the same time in indicates other objectionable qualities, such as susceptibility to changes of temperature. A considerable degree of ductility is required in most sheet asphalt specifications, but engineers conversant with the the most successful practice in asphaltic concrete work agree that ductility is not an essential quality.

It may be conservatively said that good work can be done with either ductile or non-ductile asphalt, with simplicity of manipulation somewhat in favor of the non-ductile material.

These statements, however, are not intended to dispose of the question of which is the most durable under heavy traffic or to suggest that one class be chosen in all cases to the exclusion of the other.

Successful foundations may be made of either concrete or macadam. Where traffic is comparatively heavy, unquestionably the concrete base has the advantage of greater rigidity, more "inherent stability." On the other hand, macadam in some cases may be the more advantageous to use. It does not have to stand so long to set up before you can apply the wearing surface, consequently the street is not out of service so long, and if you are working over an old street, where there is some old macadam in a more or less worn-out condition, frequently considerable economy can be effected by using a macadam base. This is a question for the engineer to work out in each case.

One principle may be laid down to apply to either concrete or macadam base. Don't finish it up smooth, but leave the surface rough and grainy, so the paving material may be forced into the interstices. The specifications promulgated by the Association for Standardizing Paving Specifications go into the question of preparing the base pretty well.

Some litigation was had in Chicago with the Bitulithic Co., of which the following is a brief statement:

In 1909, when the South Park Commissioners let a contract for paving Michigan Ave. from 12th to Jackson St. with asphaltic concrete, the Bitulithic Co. sought an injunction in the federal circuit court. The injunction was denied by one judge, but under some legal technicality the case was reopened before another federal judge in the same circuit. Injunction was again denied, and as this left the case before the court in such a shape that any further action would have to be a trial on its merits, the complainants dismissed the complaint and no attempt has been made since to renew it.

To reach this partial conclusion in the courts required over a year, and of course before then the contract over which the litigation started was completed and the pavement in use. The South Park Commissioners, however, had proceeded to lay other considerable quantities of similar pavement and if good grounds for an injunction had been shown the result would have been to force them to come to terms with the Bitulithic Co., and account for royalties on both past and future work.

Up to date, considerably over 1,000,000 sq.yd. of asphaltic concrete pavement has been laid in Chicago and the royalties would amount to a considerable sum.

It is unfortunate the Chicago case could not have been fought to a conclusion on its merits, as this might have made it possible to secure a Supreme Court decision on the "Bitulithic" patents and determine just what of their claims are valid. Several other decisions have been rendered in litigation over this patent, which are more or less conflicting, some in their favor and others contrary. It has been the policy of the patentees not to push any case to a conclusion where the preliminary rounds have been against them. For this reason it has been impossible to get an appeal into the Supreme Court.

A Horseshoe That Will Minimize Road Damage, while at the same time providing a satisfactory foothold for horses, is the subject of a competition now being conducted by the Roads Improvement Association (Wallace E. Riche, General Secretary, 15 Dartmouth St., Westminster, London, S. W.). A price of £100 ($487) has been offered. On the Judges' Committee are the well known British engineers, H. Percy Boulnois and Col. R. B. Crompton. Copies of the rules governing the competition can be obtained by sending a 1d. British postage stamp to the address given above.

Progress in the City-Manager Movement is reported as follows by H. S. Gilbertson, Executive Secretary of the National Short Ballot Organization, 363 Fourth Ave., New York City:

The proposed charter of Youngstown, Ohio, to be voted on July 22, specifies that the city manager must be a resident of the city at least five years at the time of his appointment. The commission now in session in Springfield, Ohio, has decided in favor of the plan, and it is reported that the one in Amherst, Ohio, will take similar action.

The city-manager plan is incorporated in the proposed charter of Douglas, Ariz., and will probably be incorporated in those of Bisbee and Phœnix. The charter which has been prepared by the Citizens Committee of La Grande, Ore., is of the city-manager type. The commissioners at Cadillac, Mich., have decided in favor of the city-manager form.

A Reinforced-Concrete Double-Deck Water Tank

A reinforced-concrete water tank of unusual design has been built recently at the plant of the Burson Knitting Co., of Rockford, Ill., its special feature being that it is a double-deck structure consisting of two superimposed tanks, the upper of which is carried by a girder frame

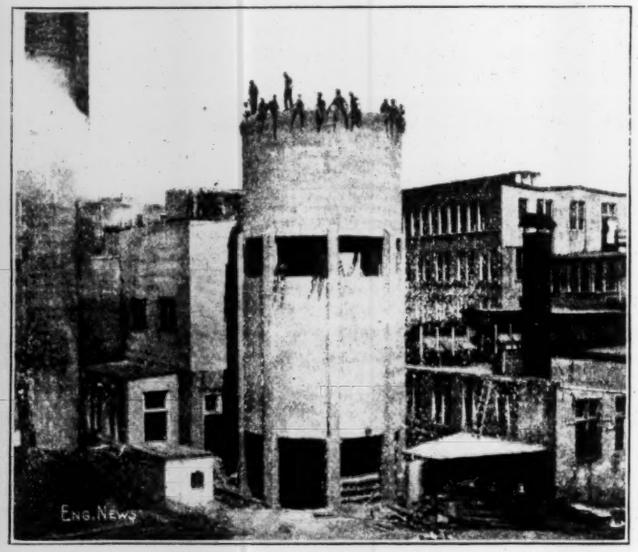

FIG. 1. DOUBLE-DECK WATER TANK FOR THE BURSON KNITTING CO., ROCKFORD, ILL.
(C. C. Stowell, Engineer, Rockford, Ill.)

on columns incorporated in the shell of the lower tank. This structure was designed by C. C. Stowell, contracting engineer, of Rockford, and was built under his direct supervision. The general design is shown in Figs. 1 and 2, and we are indebted to Mr. Stowell for plans and other information.

The company obtains water for its boilers and for washing its goods from artesian wells. The washing water is treated in a softening plant of ample capacity, but the storage capacity for the treated water was inadequate. It was desired to provide a supply of artesian water with constant head, and at such elevation that it would flow through the softening plant to a storage tank so placed that the washers could be supplied by gravity. The available space was limited, and it was decided, therefore, to place the supply tank for the raw water above the storage tank for the treated water, and to put a pump pit beneath the storage tank (Fig. 2). The water is pumped from the wells to the pump pit, from which it is raised to the supply tank by an electrically controlled pump which keeps the tank filled within fixed limits.

The ground on which the tanks were to be placed had been reclaimed from the river by filling, mostly with cinders. The surface of the ground was only 3 or 4 ft. above the river and was saturated with water. In order to obtain a secure foundation, cement grout was forced into the cinder fill by gravity to a depth of 8 ft. below the footing level of the tank. A portion of the fill, however, was found to be clay, into which the cement grout could not penetrate. In this portion, "sand piles" were used. A stick of timber was driven into the clay to a depth of 6 or 8 ft. by means of a maul and ram, and the hole was filled with sand for 25% of its depth. A slightly smaller timber was then driven into the sand to spread this and farther compress the clay. This process was repeated until the hole was filled. The "sand piles" were placed about 2½ ft. apart.

For the grouting, water was first poured into the pipes to wash and settle the material, after which a grout of neat cement (no sand being used) was poured into the pipes from an elevated hopper.

The footing was designed to carry 1½ tons per sq.ft. to the ground, and was so placed as to distribute the area equally about each column. It consists of a concrete ring 8½ ft. wide on the base, with an outside diameter of 31 ft. 6 in. This is reinforced at the bottom by eight rings of 1¼-in. rods, across which ¾-in. rods are laid in a radial position. The footing ring of concrete narrows to a top width of 18 in., which is the width of the columns. There are eight of these columns, 18x24 in. at the base and reduced to 18x18 in. in the upper portion.

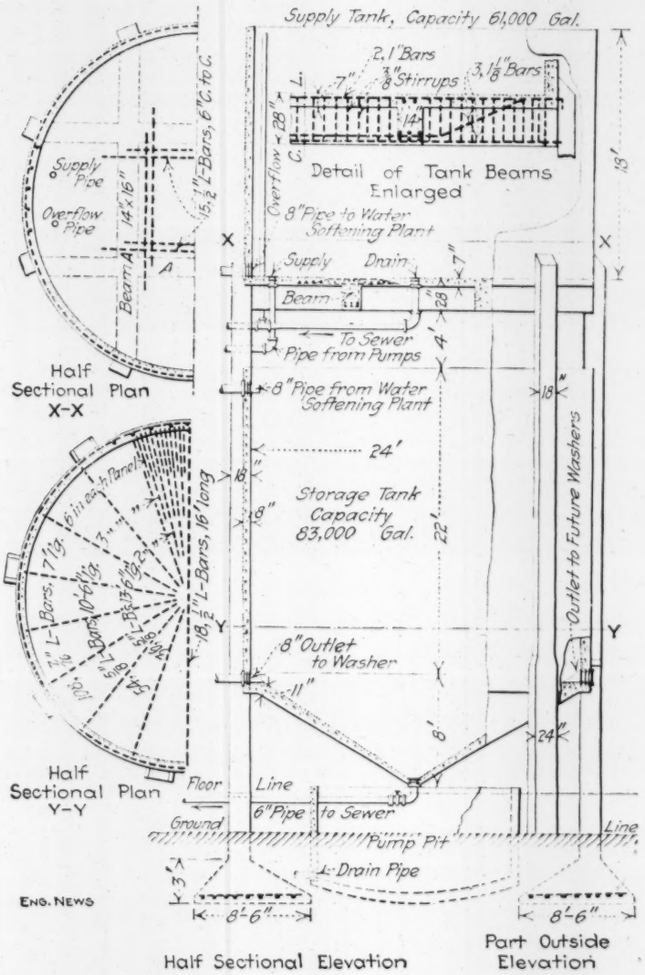

FIG. 2. DOUBLE-DECK TANK OF REINFORCED CONCRETE

The storage tank, of 83,000 gal. capacity, is between these columns, and its 8-in. shell is formed as a part of their thickness. Its cylindrical portion is 24 ft. inside diameter and 22 ft. deep, beneath which is a conical bottom 8 ft. deep for the collection of any dirt or sediment. The outlet to the washers is at the base of the cylindrical portion, while a 6-in. blowout pipe to the sewer is at the bottom of the conical base.

The columns extend about 6½ ft. above the supply

tank and are connected by four cross-beams 14x28 in., carrying a 7-in. slab which forms the base of the upper or supply tank. This tank is 24 ft. diameter and 18 ft. high, with a capacity of 61,000 gal.

The steel reinforcement of the tank walls consists of a row of ⅝-in. vertical bars and horizontal rings of bars outside of the verticals. The vertical bars are spaced five between each pair of columns in the lower tank, and 18 in. apart in the upper tank. The rings vary from ⅝-in. to ½-in. bars, with a spacing which ranges from 5¾ in. to 9 in. in the lower tank and from 6¾ to 8 in. in the upper tank. The conical bottom of the lower tank has rings and radial bars, the upper ends of the latter being turned up vertically to lie within the cylindrical shell. The bottom of the upper tank has two layers of ½-in. bars, 6-in. spacing, laid at right angles, and bent up over the tops of the beams. These beams have top and bottom bars, shear bars, and transverse stirrups. All bars are of square cold-twisted steel.

The concrete was a 1:2:4 mixture, using gravel which passed through a 1-in. screen and was retained on a ¼-in. screen. It was mixed by hand to a very wet consistency. The openings for pipes were made by threading nipples the length corresponding to the thickness of the concrete in which they were to be placed, and screwing disks on the nipples, one on each end and one in the middle. The outer disks were tapped for machine bolts, by means of which the flanges of pipes were to be attached. The middle disk was for the purpose of insuring a watertight joint about the nipple. Additional reinforcing was placed around each opening and in the top of each tank. Mr. Stowell states that while theoretically no reinforcing is needed at the top of the tanks, it has been his experience that checks and cracks usually start at the edges of work similar to this. He has made it a practice, therefore, to carefully reinforce this point. The reinforcing in the vertical walls of the tanks was placed near the outside of the wall, with the idea that if any elongation should take place in the rods, the inner surface of the wall would be less affected than it would be if they were placed at the middle.

The forms were of wood; the portion for the tank walls was made in two pieces, for the inner and outer shells. The forms for the columns were independent of the outer shell and the segments for the shell were moved up between the columns. The inside and outside forms were 4 ft. and 6 ft. high respectively. When the work on the tanks proper began, it was carried on continuously until the tank was completed. The forms were moved up by placing screw jacks under them and forcing them along as desired. The inside forms of the bottom cone were held in place by braces while concreting.

The construction plant consisted of a small friction hoist driven by an electric motor located on the ground, and a portable derrick mounted on a tower which was carried up above the falsework supporting the forms, as shown in Fig. 3. The concrete was placed in a drop-bottom bucket (holding 6 cu.ft.) and hoisted by a ⅜-in. cable operated by the derrick (manufactured by John L. Taylor, of Chicago). This swung the bucket over a hopper, from which it was distributed by means of a swinging pipe chute to such part of the work as was desired. Fig. 4 shows the end of the chute in place for concreting the conical bottom, the inner form of which is held down by the braces. The derrick proved to be very satisfactory, and it handled all the reinforcing and falsework. No staging was used in the tanks. The working platform was carried on the inner form, making it possible to have it always at a convenient height on which to work.

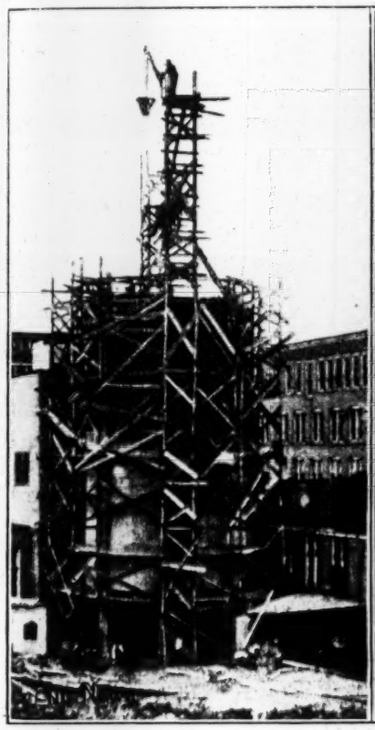

FIG. 3. FALSEWORK WITH FORMS AND HOISTING TOWER FOR REINFORCED-CONCRETE TANK

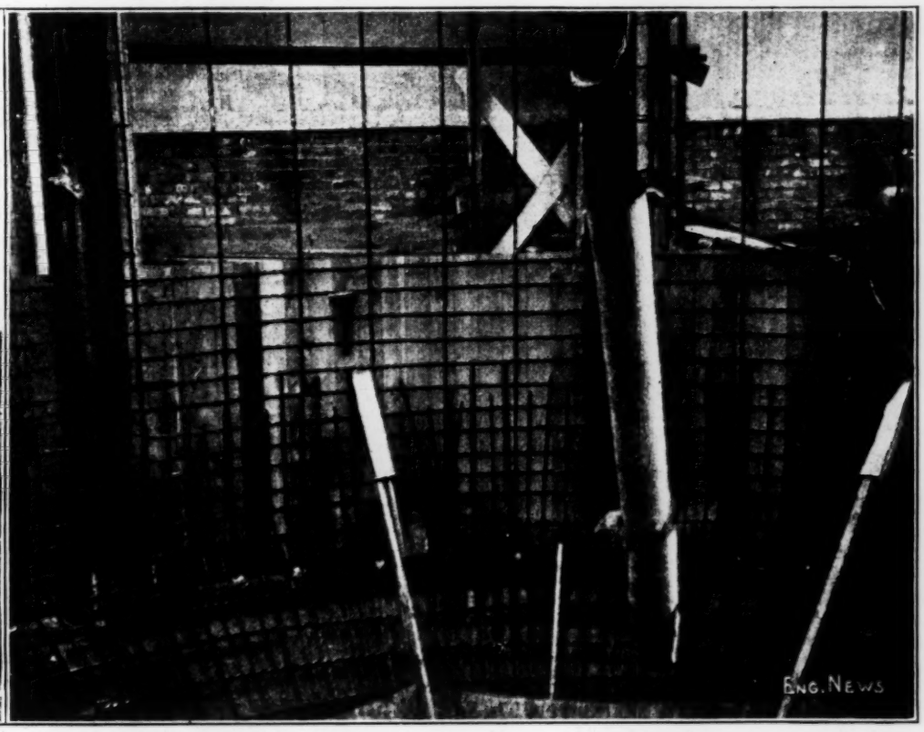

FIG. 4. INTERIOR OF CONICAL BASE OF TANK, SHOWING THE STEEL REINFORCEMENT AND THE SPOUT OR CHUTE FOR DEPOSITING THE CONCRETE

New Railway Stations and Six-Track Tunnels at Brussels (Belgium)

The Belgian State Railways have two groups of lines entering the city of Brussels from the north and the south and terminating at two pasenger terminal stations about two miles apart, at opposite ends of the city. A belt line, serving mainly for freight traffic, connects the two groups of railways, but this involves a run of about eight miles between the stations and the arrangement is very inconvenient for through trains. A few years ago it was decided to connect these two terminals by an

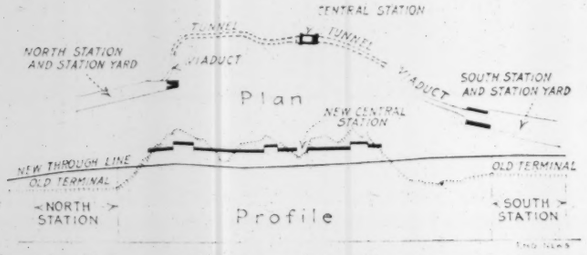

FIG. 1. PLAN AND PROFILE OF RAILWAY TO CONNECT THE TWO TERMINAL STATIONS AT BRUSSELS, BELGIUM

underground line across the city and to build a new central station on this line. At the same time additional belt lines were provided around the city to provide new facilities for through freight traffic and to keep this traffic clear of the city lines. In 1900, there was a very exceptional congestion of traffic due to the limited station facilities, the growth of suburban traffic and the inconvenient method of handling through trains. As a result of this, a commission was appointed in 1901, which adopted plans submitted by Mr. Bruneel. These are now being carried out and are described briefly by Mr. Wiener in the "Revue Generale des Chemins de Fer."

The first question was whether to enlarge or remove the existing stations. At that time the North Station had 13 tracks and the South Station had eight tracks. It was proposed at first to enlarge the stations to provide for 20 and 12 tracks respectively, with platforms 1000 ft. long and 26 ft. wide. The total cost of these changes, however, would have been some $10,000,000, which was considered out of the question. Other plans were: (1) to enlarge the stations for main-line traffic only, and to carry the suburban trains on an elevated terminal loop in each station; (2) to build a new station 2½ miles from the center of the city; (3) to divert the two groups of lines to a new terminal station between the old stations. This last plan would have added about two miles to the run of all trains, and have perpetuated the disadvantages of the terminal or dead-end type of station.

The connection of the two stations to convert the two dead-end lines into a through line had been the subject of numerous projects, and it was decided to adopt this general plan. The connecting line might be carried around the central section of the city, but this would have involved interference with the parks and the street system. As this central section is on elevated ground, a direct line could be built almost entirely in tunnel, and this route was adopted. Further, it was desired to keep the tracks as near the surface as possible, for the sake of accessibility of the new Central Station, and this necessitated raising the grade about 23 ft. through the old stations. The plan and profile are shown in Fig. 1. This arrangement is advantageous in allowing the line to cross streets on viaducts, and in eliminating former street crossings at grade at the approaches to the stations. The grades do not exceed 0.3%. Both the North and South stations are retained (but enlarged and rebuilt), the new Central Station affording additional facilities.

Approaching from the north, four separate double-track lines converge at the throat of the new North Station, and diverge to ten platform tracks with six platforms 1000 ft. long and 25 to 30 ft. wide. Baggage will be handled in a transverse subway, with elevators to the platforms. Beyond this station there are six tracks for the line to the Central Station, which is approached by 3300 ft. of tunnel, but is itself in the open. This station has eight tracks with platforms about 1000 ft. long. A six-track tunnel of 1200 ft., followed by a viaduct, brings the line to the new elevated site of the South Station. This has ten through tracks, besides stub tracks for mail and baggage cars, milk trains, etc. The tracks converge to a throat with crossover and slip-switch connections, beyond which are 12 main tracks to the next station where coach yards and engine facilities will be provided, as at the approach to the North Station.

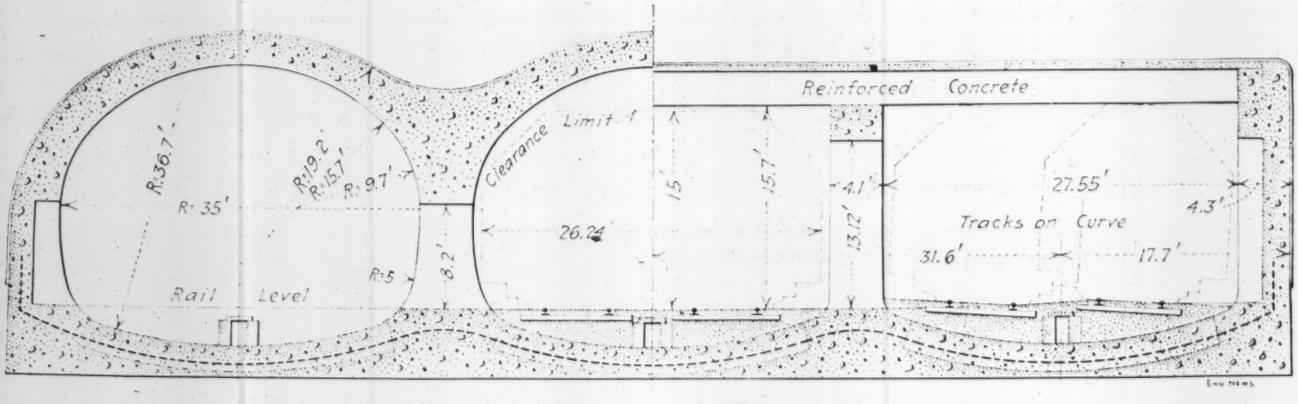

FIG. 2. CROSS-SECTION OF TRIPLE (SIX-TRACK) TUNNEL AT BRUSSELS
(Partly of circular and partly of rectangular section.)

The soil is largely sand, including quicksand in places, but the invert will not be more than 10 ft. below the ground-water level, which will be lowered by means of drainage, as has been done at other tunnels, including the Metropolitan Ry. tunnels at Paris and Berlin. The tunnel (Fig. 2) will have three double-track sections, each with an arched roof in the deeper portion and a

flat roof of reinforced-concrete girders in the shallower portion.

The construction of the Central Station involves extensive changes in an old and congested district, and the construction of new streets and approaches. The state has undertaken to acquire the property and to guarantee the city against loss due to real-estate operations. The rail level will be practically at the level of the old streets, the present grade being raised. The station building is across and at right angles to the tracks, fronting on one of the new streets, and behind it is a broad concourse with stairways to the platforms. It is to be noted that this will be simply an intermediate or local station for passengers, no baggage being handled here, although the building has been designed with a view to handling baggage at some future time, if this should be found desirable.

Trains will be hauled between the two main stations by electric locomotives, without detaching the steam locomotives. For this purpose special stub or service tracks for the electric locomotives are provided adjacent to the end of each platform track. The trains will weigh about 350 tons, or nearly 100 tons with the steam locomotive and tender. The electric traction system has not yet been decided upon.

The work was to be completed by January, 1915, but it is probable that an extension of time will be necessary. The cost of the triple tunnels is estimated at $274 per ft., and that of the stations at $2 per sq.ft. The total cost is estimated as follows:

Alterations at the North Station	$1,928,000
North Station to Central Station	2,117,000
Central Station	3,631,000
Central Station to South Station	2,501,000
Alterations at the South Station	1,134,000
Streets, sewers, etc.	289,000
	$11,600,000

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Highway Administration in Illinois has been advanced materially under the new law which was signed recently by the governor, and will include state aid and county supervision. There has been for some years a State Highway Commission, but the new law creates a State Highway Department, consisting of a commission of three members, a state highway engineer, an assistant state highway engineer, and such other employees as the work may demand. The members of the State Highway Commission will receive an annual salary of $3500 each, and they are to be appointed by the governor within 30 days. The state highway engineer and assistant state highway engineer are also to be appointed by the governor, and to receive salaries of $4000 and $2500 per year respectively. All other employees of the department are under the civil-service law of the state.

The law also provides for a county superintendent of highways in each county, to be chosen as follows: The county board is to submit a list of from three to five names to the State Highway Commission; these candidates will be given a qualifying examination and all who pass will be certified to the board as eligibles for the position. The county board may then name anyone from this list as county superintendent of highways, at a salary to be determined by the county board. The county superintendent will be the deputy of the State Highway Engineer and will take care of the engineering on county and township bridge work and all state-aid road work in his county.

The law provides for state aid in the construction of certain roads in each county to be known as state-aid roads. On such roads, half the cost of improvement will be paid by the state and the other half by the county. In order to get in line for state aid, each county must file a map on which are shown those roads which are to be state-aid roads. It must also take the necessary steps to finance the county's half of the cost of construction, and then make application for state aid.

The old State Highway Commission continues until the new one is appointed, but all other officials and employees of the department were temporarily reappointed on July 1, so that the work is going on without interruption.

The Tour of the American Society of Mechanical Engineers in Germany

(Editorial Correspondence)

The joint meeting of the American Society of Mechanical Engineers with the Verein Deutscher Ingenieure at Leipzig, Germany, on June 23 and 24, 1913, was followed by fourteen days' touring in Germany. Factories, shops, harbors and other points of technical interest were visited at all the large cities on the route. Beyond this the trip was bare of engineering activities. But the whole tour was characterized, on the part of the Germans, by hospitality so full and cordial, and on both sides by such warm fraternization, as to make the occasion memorable and significant. A climax—in one sense—was reached near the close of the trip, at Munich, where in the genial atmosphere of the Hofbräuhaus some enthusiastic Americans started an attempt to form a "New York Bezirksverein" of the Verein Deutscher Ingenieure. More important was the very evident impetus given at Munich—where the wonderful Deutsches Museum of Science and Engineering was shown—to the idea that a similar record-collection must be formed also in the United States, to chronicle the world's advances in the arts and industries.

To give everyone his due, let it be said that the "Deutschlandreise" gave splendid proof of the American contingent's powers of endurance. The cities, the "Bezirksvereine" and the industries visited poured out substantial welcome in a continuous flood, with an effect which to the participants was like the strain of a three-mile run. Yet they held out steadily and soberly, in spite of scanty sleep and the cumulative influence of some 28 banquets and formal luncheons in 18 days.

That Germany has attained an industrial development of phenomenal character was made clear to the visiting party in the shops visited, in the statistical summaries and the self-conscious though modest speeches delivered by representatives of the hosts, and in the general evidences of universal prosperity of the nation. The tour was made in a country, which, with two-thirds of the population of the United States, concentrated on a small area, supplies nearly its whole demand for food products from its own agri ulture; which displays intensive cultivation of the soil, both in the country districts and close up to the city areas; which carries a vast commerce on its rivers and canals, and is building additional canals at a rapid rate; which possesses the largest inland harbors of the world, and is extending these harbors with great enterprise; and which has in its prime industrial district—the Düsseldorf-Essen region—maximum shop production and home-factory production almost side by side, and in equal flower. Observation of these facts, and of the economical and orderly habits of the people, conveyed a convincing impression of Germany's present-day industrial strength.

As a record of the tour the following list is given of cities visited and inspection trips made, subsequent to the stay at Hamburg and Leipzig, reported in an earlier issue.

Dresden was visited June 25 and 26. An excursion to the Bastei, at Rathen, consumed much of the available time, and the many art treasures of the city engaged further interest. The laboratories of the Dresden Tech-

nische Hochschule, and the shops of Seidel & Naumann (bicycles, sewing machines, etc.) were visited.

Berlin, June 27 and 28, gave occasion to visit the cable plant of the Allgemeine Elektricitäts Gesellschaft (Oberschöneweide), the Siemens & Halske and Siemens-Schuckert electrical works (Nonnendamm), the plant of the Bergmann Elektricitäts Werke A. G. (Rosenthal), the machine-tool plant of Ludwig Loewe & Co. A. G., and the locomotive plant of A. Borsig (Tegel). Small groups also visited the Material prüfungsamt (Lichterfelde), the Patent office, the Technische Hochschule (Charlottenburg) and other institutions. One of the noteworthy events of the Berlin stay was a visit to the Carl von Siemens estate on the Havel, preceded by a steamer trip from Wannsee; military aviators flying over the grounds, and song renderings by the Potsdam male choral society, added to the occasion.

Düsseldorf, the center of Germany's "Pittsburgh district," occupied June 30 to July 1. The Krupp works at Essen showed but little of their elaborate establishment, but very full inspection was made of numerous other plants: the crane and hoist works of the Deutsche Maschinenfabrik A. G. at Duisburg and Benrath, the engine works of Thyssen & Co. A. G., at Mülheim-on-Ruhr,* the Gutehoffnungshütte steel works at Oberhausen, the Rheinische Stahlwerke (Meiderich), Haniel & Lueg (Grafenberg), Ernst Schiess A. G. machine-tool plant (Düsseldorf), the steel works of the Becker A. G. (Willich), and smaller plants in the Solingen neighborhood. The Düsseldorf branch of the Verein Deutscher Ingenieure distinguished itself by an artistic and highly elaborate stage production, in which a lifelike model of a redhot ingot was forged into a ring inclosing the emblems of the American Society and the Verein.

The party now moved up the Rhine to Cologne. The two great engine works of Gasmotorenfabrik Deutz, at Deutz, and Maschinenbauanstalt Humboldt, at Kalk, were visited, besides the large chemical works of Farbenzabriken vormals F. Bayer & Co., at Leverkusen. The Deutz works, the "home of the Otto cycle," showed, among other things, many Diesel engines of all sizes in course of construction or operating, and some engines of a modified Diesel type which avoids use of high-pressure air injection by means of an oil spray cage.

From Cologne the party went to Frankfurt a. M., but here a day was devoted to a steamer trip up the Rhine through the highlands section, passing the ruins of the robber barons' castles and the many miles of vineyards which keep the river's fame alive, and passing also a great number of tows of loaded barges carrying coal and other freight up the river. This trip from Coblenz to Rüdesheim formed one of the best-appreciated parts of the entire tour.

In Frankfurt a. M., July 4, visits were paid to various municipal works, including the harbor, the refuse incinerator, etc., and to the shops of Adlerwerke vormals H. Kleyer A. G., Pokorny & Wittekind Maschinenbau A. G., Simon Bühler & Baumann, and Voigt & Haeffner A. G. The well-restored Roman camp, the Saalburg, above Homburg, was also visited by many of the party. In the evening a Fourth of July banquet with fireworks was held in the Homburg Kurhaus.

Mannheim, at the junction of the Rhine and Neckar, a keypoint of Rhine freight shipping, was the headquarters July 5 and 6. The second day, however, was used for a trip to the Heidelberg Schloss, with a barge trip down the Neckar and red-fire illumination of the castle a gorgeous finale. The works visited at Mannheim included the Sulzer Bros. plant at Ludwigshafen, the shop of Brown, Boveri & Co. A. G. at Käferthal, the Maschinenfabrik Heinrich Lanz, Benz & Co.'s great plant, the Heidelberg Portland Cement Werke, and the Ludwigshafen flour mill. The Mannheim harbor, next in size to that of Ruhrort (near Düsseldorf), was shown in a steamer trip, and this inspection gave some explanation of the 5,700,000 tons inbound traffic per year and the $35,000,000 exports to the United States of which the city boasts.

München, the terminus of the tour, was reached July 7, and the party broke up with a splendid banquet in the five-century-old Altes Rathaus on the evening of the 8th. The chief event of technical interest was a trip through the Deutsches Museum, and a visit to the half-completed new building of the museum, laid out on an enormous scale. The president of the Verein Deutscher Ingenieure, Dr. Oskar von Miller, was the creator and mainstay of this enterprise. The closing ceremony fittingly turned into an ovation for Dr. von Miller and for the Verein's secretarial representative, Dr. Conrad Matschoss, who had accompanied the party during the entire three weeks from Plymouth to Munich. Renewed invitations to the Verein to visit America in 1915, on the occasion of the Panama-Pacific Exposition were given and accepted.

The entire tour was notable not only for the generous hospitality of the various local sections of the Verein (Bezirksvereine), but equally for the openhanded welcome given the party by the authorities of the various cities visited. The musical and artistic spirit of the German race was also amply displayed in concerts, singing performances by men's choirs (chief among them the Kölner Männer Gesangverein), boy choirs (including the noted Thomaner choir at Leipzig) and mixed choruses, and in the splendid decorative setting given to the main social events of the tour. It will be hard indeed to make the 1915 "Amerikareise" of the Verein—if it materializes—equal this trip of the American Society of Mechanical Engineers through Germany.

F. E. S.

*Here a Holzwarth gas turbine was shown in operation; it was stated to be a working success.

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A Public-Health Exhibit Train will be run through the state of Michigan, under the joint auspices of the State Board of Health and the Dairy and Food Department of the State Agricultural College. The train will start on Aug. 4. It will consist of an engine, at least two exhibit cars and a Pullman car for living purposes. The following extracts from a note describing the train, which appeared in the May Bulletin of the Michigan State Board of Health, gives further particulars of the exhibit:

The exhibit will be grouped under various heads, such as Tuberculosis, Typhoid Fever, Summer Resorts, Milk, Water, Flies, Market Sanitation, Pure Foods, Pure Drugs, Fake Medicines, Baby Conservation and many other interesting subjects. Every bit of wall space and table room in these coaches will be covered with charts and models, nearly all of which are self-explanatory.

Upon our former trip, very frequently the crowds were so large that not all of the people were able to get within the exhibit coach during the time allotted to the individual stations. This time our schedule is much better. It provides for longer stops and a great many more stations.

Along this route we are arranging stops at practically all towns having a population of 500 or more. The length of time spent at each station depends, of course, upon the number of people to be accommodated.

This is an exhibit which the public cannot afford to miss. Great quantities of literature will be distributed. In each town where over-night stops are made, lantern-slide and moving-picture demonstrations will be given. In fact, this will be a public-health evangelistic tour and it is certain to be of great value to each community.

The Construction and Use of Empiric Formulas

By Cyrus T. Brady, Jr.*

The great advances made in the methods of design of structures, due to the possibility of computing the distribution of load in statically indeterminate frames by the principle of least work, the thorough establishment of the theory of the elastic arch and other applications of the higher mechanics, these very advances have had a tendency to blind us to the essentially empiric nature of the design of the majority (in value at least) of engineering construction. Retaining walls, bridge abutments, pile foundations, earthwork, tunnel linings, the impact coefficient in bridge design, are only a few of the cases with which science has not yet been able to deal with precision.

Formulas for, or methods of designing these structures usually meet with two kinds of reception, diametrically opposed. Either they are passed by without further consideration or are accepted instantly with a trusting confidence in the name and reputation of their author. Sometimes on merely observing that the formula agrees in two or three cases with known values, it is trusted implicitly. This attitude is quite as objectionable as that which discards the formula without consideration, and both are probably due to the prevalent unfamiliarity with the actual construction of such formulas.

Methods of Constructing Formulas

The writer has seen other engineers many times surprised at the accuracy with which empiric formulas of his own have agreed with known values and he has been led to believe that the methods of constructing them may be of general interest. There are three general classes of these formulas: (1) The essentially empiric, where the formula is shaped to fit certain observed values without regard to the law governing their variation; (2) the rational formula with coefficients modified or simplified empirically; (3) the semirational formula where the coefficients must be determined entirely empirically.

It is in treating the first class that the greatest difficulties arise, but systematic investigation will abolish many of them. There are cases where the values are so irregular that no formula can be made to fit them, or where different formulas must be found between different limiting values of the variables. It is, of course, preferable to express all the values by one formula and as the partial formulas are constructed in exactly the same way the following suggestions apply to both types. It may be stated at the outset that familiarity with analytic geometry, a little ingenuity and practice will greatly aid in such work.

Let us assume that there is but one variable. Many times an equation of the form $y = a + bx^n$ will fit the observations. In this equation a is the value of the function when $x = 0$ and is thus easily found. Plotting the values of $(y - a)$ and of x on logarithmic paper the resulting curve will be a straight line cutting the Y axis at the value of b and with an inclination to the X axis such that the tangent of the included angle will equal n. Plotting the logarithms of the same quantities on ordinary cross-section paper will give the same results. This method is well known and easy of application, but it usually leads to a fractional value of n, which in practice is a nuisance.

The same formula may be derived by substituting three pairs of values of y and x in the general equation and solving simultaneously for a, b and n. This method can of course be applied whenever the general form of the equation is known, provided there are as many sets of values of the function and variables as there are constants in the general form of the equation. It is, however, unusual to know the general form.

A more simple and general method follows: Write down in parallel columns values of the function and the variables with, if possible, equal increments for the latter (these can sometimes be obtained by interpolation). If a can be determined subtract it from each y and write a new column of $(y - a)$. Observe the change in this quantity when x is two, three, four and eight times one of its values. Thus, if n be 2, 3, $\frac{1}{2}$, $\frac{1}{3}$, $\frac{3}{2}$, $\frac{2}{3}$, etc., its value can be found very quickly. Knowing n, b is then found.

In case a cannot be determined, write down a column of the successive differences of the function for equal increments of the variable. These first differences will in general be unequal. Find then the second differences (i.e., successive differences of the first differences) and so on until the rth differences are all equal or vary only slightly irregularly. Call this equal difference d. Then $n = r$ and $b = d$ divided by the product of the numbers from 1 to r. Knowing b and n, a is found from any value of y. This method does not apply simply when n is not a positive whole number. When the pth differences are unequal and increasing and the $(p + 1)$th differences are unequal and decreasing, a fractional value of n between p and $(p + 1)$ is indicated. When the first differences are unequal and decreasing a negative value of n is indicated. It frequently happens that slight errors of observation cause the rth differences to be very irregular, although the values of the function are closely represented by the general equation. Hence, the method is not of much practical use, but the study of the increments usually suggests lines of attack.

An effort should always be made to determine any constants of algebraic addition or multiplication and if possible these should be eliminated from y so that only the variable part of the equation need be considered. Having written down the values of the function and opposite them the values of the variable, observe whether x^2, x^3, $\sqrt{x}$, $\sqrt[3]{x}$ or some other simple function does not vary approximately with y. If so write down a column of the differences between the value of the assumed function and the observed values, or a column of the quotient obtained by dividing y by the assumed function. On studying this column some law of variation may be perceived. Observe the effect of adding or subtracting a small amount in the assumed function. For example, $y = \sqrt{x + 2}$, gives much higher values of y for low x than $y = \sqrt{x}$, but for high x, y will be nearly equal. Having thus approximated to the observed values, treat the discrepancies as a new problem and approximate to them, of course going to no greater accuracy than the observations warrant.

Study the variation of simple equations. For example $y = a + bx + cx^2$ may be made to increase to a certain

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value and then fall rapidly away, by giving c a negative value, or it may increase slowly and then rapidly by giving c a low value, and *vice versa*. Many empiric formulas can be expressed by this equation. Another which can be given useful forms is $y = \frac{a + x^n}{b + cx^m}$.

When there are two or more variables, the case is much

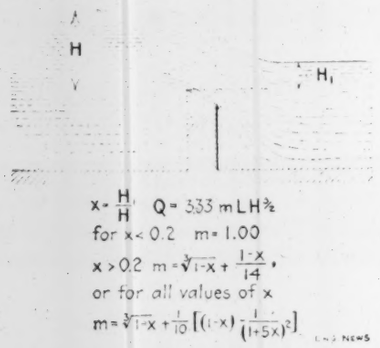


FIG. 1. HERSCHEL'S FORMULA FOR SUBMERGED WEIRS AS AMENDED BY BRADY

more difficult. If the observations are sufficient in number, the variation caused by one variable when the others are constant can be studied for each and the equation thus found. If the observations are insufficient the general form of the equation must be assumed and the coefficients determined by solving simultaneously.

It is frequently possible to make equations of very different form give very similar results. For example, Stephenson's formula gives the height of waves H, in feet caused by the wind:

$$H = 1.5 \sqrt{F} + (2.5 - \sqrt[4]{F})$$

F is the fetch in miles (or longest straight sweep of the wind over the water surface). For F above 30 the second term of this formula is omitted, nevertheless the following table gives values determined by the second term and by the following function $\frac{40 - F}{16 + 8.65\sqrt{F}}$

F	0	1	10	40	100	300	500
Stephenson	2.5	1.5	0.7	0.0	−0.7	−1.7	−2.2
New Form	2.5	1.5	0.7	0.0	−0.6	−1.6	−2.2

The longer form is perhaps no improvement from the point of view of simplicity but it is interesting as showing a remarkable agreement within wide limits.

APPLICATION OF FORMULAS

This brings us to the question of the limits of application of empiric formulas and this is the weak point in their use. It is probably safe to say that most of them must not be applied for values of the variable more than 10 or 20% beyond the range of the observations and this both limits their use and emphasizes the necessity of knowing before applying them the exact limits of observation on which they are based. For example, in a place where the maximum daily rainfall is 8 in. the following formulas have been used with good results for the number of inches R fallen during any number of hours T from 1 to 24.

$$R_a = \sqrt{\frac{T}{24}} \times 8 = 2.77\sqrt[3]{T}$$

$$R_b = 1.53\sqrt{T} + 2$$

T	0	½	1	4	8	16	24	48
Ra	0	2.2	2.77	4.4	5.5	7.0	8.0	10.1
Rb	2.2	2.5	2.72	3.9	5.0	6.6	8.0	11.1

It is seen that they agree very well for the range for which they are used but that below one hour R_b is quite useless.

FRANCIS WEIR FORMULA—An example will now be given of the construction of a complicated essentially empiric formula. In 1885 Herschel investigated the subject of submerged weir discharge in the light of all the better experiments and adopted, as a working formula for the discharge, the familiar Francis formula in which he substituted for the head H a fictitious head nH. n is a coefficient varying irregularly with the ratio x (see Fig. 1) and was given by Herschel in a long table derived from experiment. In the writer's formula given below Fig. 1, m is merely Herschel's n raised to the $^3/_2$ power and is given in column (2) of Table I.

TABLE I

(1) x	(2) m	(3) 1 − x	(4) $(1-x)^3$	(5) 1 − x − (4)	(6) $\frac{1-x}{10}$	(7) (6) − (5)	(8) $\frac{0.100}{(1+5x)^2}$
0.0	1.00	1.0	1.000	0.000	0.100	0.100	0.100
0.1	1.01	0.9	0.965	0.045	0.090	0.045	0.044
0.2	0.98	0.8	0.928	0.052	0.080	0.028	0.025
0.3	0.94	0.7	0.888	0.052	0.070	0.018	0.016
0.4	0.89	0.6	0.843	0.047	0.060	0.013	0.011
0.5	0.84	0.5	0.794	0.046	0.050	0.004	0.008
0.6	0.78	0.4	0.737	0.043	0.040	−0.003	0.006
0.7	0.70	0.3	0.669	0.031	0.030	−0.001	0.005
0.8	0.59	0.2	0.585	0.005	0.020	+0.015	0.004
0.9	0.44	0.1	0.464	−0.024	0.010		0.003
1.0	0.00	0.0	0.000	0.000	0.000		0.003

On examining the column (2) it is seen to fall from 1.00 to 0.00 while x is rising the same amount. This suggests column (3) of $(1 - x)$. We now have m and the variable varying more or less directly and by trying the effect of different powers of $(1 - x)$ we find that the cube root is closely related to m. Column (5) contains the differences between this root and column (2). For values of x greater than 0.2 these differences vary, with some regularity, with $(1 - x)$. A few trials showed that dividing $(1 - x)$ by 11 gave a series approximating the differences. Accordingly the simple formula was written. If it be remembered that for x below 0.2, m may be assumed constant at 1.00 (the error being less than 2% which is well within the probable error of application to any given case), the original lengthy table may thus be replaced by two lines.

Merely to show that one formula can be made to express the whole range of x, the analysis was carried further as given in the table. The general form $\frac{b}{(1 + ax)^2}$ was chosen in order to equal the value of $(1 - x)$ divided by 10 when $x = 0$ and which should rapidly decrease with increase of x. b is, of course, 0.10 and after a few trials the value of a determined as 5. The longer formula was thus prepared. As a comparison of the three the following table is given:

x	0.0	0.1	0.2	0.4	0.6	0.8	0.9	1.0
Herschel's n	1.00	1.01	0.98	0.89	0.78	0.59	0.44	0.00
Long Formula	1.00	1.01	0.98	0.89	0.77	0.60	0.47	0.00
Short Formula		(1.03)	0.98	0.89	0.77	0.60	0.47	0.00

DOUBLY REINFORCED-CONCRETE BEAMS—As an example of a rational formula modified empirically let us take the case of the moment of inertia of the doubly reinforced-concrete beam shown in Fig. 2.

The rational equations are long and cumbersome and since there is much uncertainty in the proper value of n it was believed they might be much simplified within an entirely allowable limit of error. The value of n was taken as 9 in accordance with the conclusions of Turneaure and Maurer founded on experiment. n thus

becomes, in a way, an empiric coefficient correcting the errors inseparable from working assumptions. d_1 was taken constant at $0.1d$. Then for various values of p_t and p_c between 0.00 and 0.02 the moment was computed.

The long bracketed coefficient of bd^3 reduced each time to an equation of the form $a = r + sp_t + tp_c$. The extreme range of r was from 0.083 to 0.090, of s from 1.7 to 3.0, of t from 1.3 to 2.2. Averages of these were taken and the formula given was written.

The error is almost always considerably less than that occasioned by changing n to 8 or 10, and hence the formula may be confidently applied in the case of deflection of beams. Computing similar formulas for other values of n, the variation appeared regular enough to justify the general simplified form given in Fig. 2.

QUAY-WHARF DESIGN—As a last example we will take a semi-rational formula with empiric coefficients.

The writer was called upon in 1912 to prepare estimates and preliminary designs of quay-wharves, etc., for coal and timber traffic, at one of the large ports of the River Paraná. As several different arrangements were to be considered where the quays and piers varied in size and position, it seemed probable that the time spent in developing a formula for the cost per square foot of the wharves would be more than made up in the more ac-

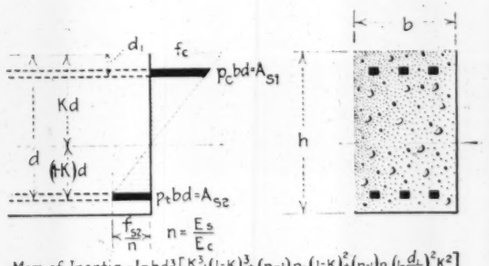


FIG. 2. MOMENT OF INERTIA OF DOUBLY REINFORCED CONCRETE BEAMS

curate comparisons thus possible without detailed designs.

The wharves were to be supported on piles and made of timber (cypress, yellow pine and quebracho, the hard, heavy and very durable wood of the country). The decks were to be 4-in. creosoted yellow pine covered with a 2-in. sheathing. The caps, stringers, floor beams, etc., under the decks were dimensioned and spaced (as closely as seemed practical) in accordance with the live load so as to stress the 4-in. deck to a reasonable, uniform figure. Hence, the deck and sheathing were common to all the types. The designs were made for definite live loads and the piles spaced so that each should receive 32,000 lb. The piles were to be driven to a total penetration of 5 m. (16.4 ft.). As there are several thousand piles in the same port driven to 7 m. (23 ft.) and taking maximum loads of over 25 tons under masonry quay walls, the figure assumed was certainly safe.

Sketch designs for two quays and a pier (which was practically the same as the two end spans of one of the quays) were prepared and the costs estimated with some care; 15% was added for superintendence and incidentals; 15%, as is usual in the Argentine, was also added for the contractor's profit. The data for these designs follow: (See Fig. 3.)

N	L	D	P
14	400	18	1.74
10	750	22	2.65
2	750	36	6.40

In considering the cost of a wharf it was seen that there were five main elements: (1) The deck; (2) the structural arrangements for supporting it; (3) the sway bracing; (4) the length of piles between deck and river bottom; (5) the length of piles in the bottom.

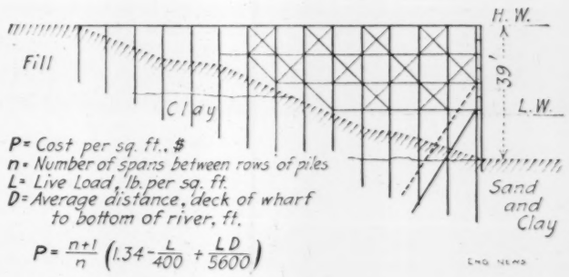


FIG. 3. COST OF PILE QUAY WHARVES

(1) The cost of the deck per sq.ft. is constant, $= a$.

(2) In one panel the area to be supported by the stringers, etc., becomes smaller as the live-load increases (since the pile loads are fixed). The scantlings are practically constant but there are more of the beams per sq. ft. as the load increases. Hence their cost $= b + cL$.

(3) The bracing of the piles increases with the depth and, similarly to the beams and stringers, with the live-load, as their scantlings are constant. Cost is therefore $= d + cLD$.

(4) The spacing of the piles is a direct function of the live-load and so, therefore, is their cost per sq.ft. The cost also depends directly on their length and in all is fLD.

(5) The cost of driving and of the length of piles in the river bottom, per sq.ft., is directly dependent on the spacing and therefore on the live-load, and equals gL.

To allow for the outer row of piles, the string piece, clamps, rubbing pieces, extra cost of work as the number of units decrease, and so on, the sum of the other items should be multiplied by $(n + 1)$ divided by n.

The sum of the five main elements equals

$$a + b + d + (c + g)\,L + (e + f)\,LD$$

or

$$A + BL + CLD$$

After dividing the calculated costs of the three actual designs by the fraction $\frac{n}{n+1}$ the quotients were set equal to the expression just given and on substituting the known values of L and D and solving simultaneously the three constants were determined.

As an indication of its accuracy, after the formula was developed, a design for another wharf was made and its cost estimated as $4.90 per sq.ft. The formula gave $5.10. The costs given by the formula are about one-third too high, largely on account of freight charges, for similar wharves in the northeastern portion of the United States. It is believed that the formula can be applied in the port for which it was derived for a range of L from 250 to 1000 and for D from 11 to 40.

FIELD AND OFFICE

A Large-Capacity Building Derrick

The accompanying view shows a very large structural-steel derrick which is now in use in the construction of the Brooklyn Arts & Sciences Building, Brooklyn, N. Y. in the contract of Wills & Marvin Co. The derrick was designed and built by the Alfred E. Norton Co., of New

A Building Derrick with 100-Ft. Boom

York City. It has, as shown in the view, a mast of 110 ft. and a boom of 100 ft., which gives it a capacity of 15 tons at the extreme end of the boom. Both mast and boom are divided, as shown in the view, into three parts and there has been provided a separate center piece for both the boom and the mast. When this shorter center piece is placed instead of the longer one shown in the view the mast length is 86 ft. and the boom length is 75 ft. With these members the derrick has a maximum capacity of 20 tons.

Both mast and boom can be separated into the three pieces shown and the whole derrick knocked down so to be shipped complete in one car.

The base of the mast is provided with a steel-socket casting and the alignment of the mast is such that one man can operate it when loaded equally as well as in a smaller derrick. It is evident that the extreme reach and capacity of this derrick make it an extremely efficient tool to be used in the construction of large-area buildings.

A Small Bascule Highway Draw Span

The average engineer is probably more familiar with the design and construction of particularly large or complex drawbridges, which are frequently described in current engineering literature, than with the details of small drawspans such as he may at any time be called upon to design. Hence, the description of a simple and economical structure designed by L. E. Moore, Associate Professor of Civil Engineering at the Massachusetts Institute of Technology, for a highway crossing of 36-ft. span, may prove valuable to many another engineer who meets a similar problem.

This drawbridge was designed for the George M. Bryne Co., of Boston, Mass., who had contracted to build a bridge extending from the mainland to Grand Island in the village of Osterville, Barnstable, Mass., to replace a wooden pile structure having a double-leaf wooden stringer drawspan. The new structure consists of four 42-ft. spans of the reinforced-concrete girder and slab type with a 36-ft. central drawspan of steel and timber construction. It is described by Prof. Moore in a paper read before the Boston Society of Civil Engineers, Feb. 19, 1913.*

The contract plans showed a sketch of a double-leaf steel drawspan similar in type to the wooden one to be replaced; but on inquiry the designer found that there was but one bridge tender. He was obliged to raise one leaf of the draw and cross over in a rowboat to lift the other, before a boat could pass. As it was desired to operate the bridge by hand the two-leaf structure was ruled out of the new design.

The type of construction adopted consists of two longitudinal steel girders supporting four steel floor-beams, upon which rest wooden stringers and a wooden plank floor of the ordinary type. The trunnions are on the mainland end of the bridge. The lift span is counterbalanced by a solid block of concrete, the upper surface of which, when the draw is closed, forms the floor of the roadway adjacent to the lift span. When the draw is opened, the concrete counterweight swings down between two concrete columns which support the bearings of the trunnions. The draw is operated by the quadrant shown in the illustration, Fig. 1, of the completed structure.

A plan, elevation and cross-section of the bridge are shown in Fig. 2. The floor consists of two layers of 2-in. planking laid on 2x14-in. hard-pine stringers, spaced 2 ft. on centers. Holes were punched in the top flanges of the floor-beams and ¾-in. lag screws driven into the bottoms of the floor stringers to hold them in position when the span is lifted. The stringers bear against the concrete counterweight, which relieves the floor-beams from any bending in a direction perpendicu-

*Journal of the Association of Engineering Societies, June, 1913.

lar to the faces of the webs when the bridge is open, and also obviates expensive bracing of the floor-beams.

The brackets supporting the outer edges of the roadway are held by single-hitch angles, and no provision

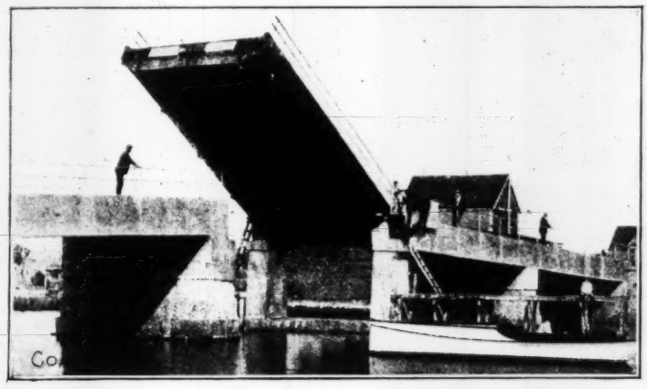


FIG. 1. A 36-FT. SPAN BASCULE HIGHWAY DRAW SPAN, BARNSTABLE, MASS.

is made for a plate connecting the top flange of the floor-beam to the top of the bracket. The tension in the connecting rivets is relied upon to hold the bracket under any load that may come upon it. Any tendency of the brackets to twist sideways is amply resisted by the wooden floor-stringers which are fastened to the brackets by lag screws, and by the floor planking which is continuous over the whole width of roadway.

The details of the girders and floor-beams are shown in Fig. 2. The trunnions on which the bridge turns are made of two 15-in. 42-lb. I-beams, which extend through a rectangular hole cut in each girder. An iron casting, or elongated separator, is fitted between the I-beams at each end, and extends from just inside the web of the plate-girders to the face of the bearing on the abutment. A shaft 7 in. in diameter is secured to this casting or separator by keys and bolts, and forms the journal or trunnion proper.

The trunnion at one end is made just long enough to extend through one bearing, and at the other end extends through two bearings. At this end there is keyed to it at its outer end a gun-metal quadrant, which is operated by a train of gears. The bearings are phosphor bronze and babbitt metal. The phosphor bronze is used in the lower half of the two boxes closest to the girder, and the babbitt is used in all the caps and the box next the quadrant. A portion of the flooring about 3 ft. wide by 4 ft. long over each end of the trunnion is removable to allow ready access to the machinery at any time.

The concrete counterweight is fan-shaped in elevation and completely envelopes the girders for the full length of the counterweight. The girders are tied together by angles at the ends carrying the counterweight, and a vertical stiffener is put on the inside of each girder in order to serve as an anchorage for the concrete block.

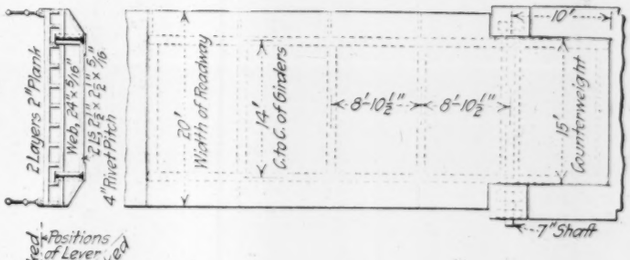
The counterweight was computed as a reinforced-concrete beam between the girders and was designed to carry the roadway loads in addition to its own weight. It is reinforced by ¾-in. bars spaced 12 in. on centers, laid on the bottom flanges of the girders. The lower edge is similarly reinforced, the bars in this case being hung on vertical rods bent over the top flanges of the girders. When the bridge is open, the counterweight and bridge occupy the position shown by the dotted lines (Fig. 2).

The solid masonry pier is cut off horizontally at a point just above high water, and two stout concrete posts are carried up at each end. These concrete posts are approximately 2 ft. 6 in. by 3 ft. 6 in. each. The bearings for the support of the drawspan rest on the top of these posts, as do the girders of the approach span.

The piers of the bridge in general are constructed of split granite with a concrete cap. In order to prevent any possibility of the draw pier splitting, three 20-in. 65-lb. I-beams were laid on top of the top course of granite to distribute the weight of the bridge. In order to protect the concrete with which the I-beams are surrounded from the action of the salt water, a belt course of granite was laid around it and thoroughly tied in by means of iron dogs. This course was laid first and used as a form for the concrete surrounding the I-beams.

To hold down the end of the drawspan opposite the counterweight, which tends to rise when any live-load comes on the top of the counterweight, it was necessary to design a locking device which would hold the end of the draw tightly to the abutment, so that it could not hammer as the load passed over it. This device is illustrated in Fig. 3. It consists of an eccentric carrying an elongated hook, which is free to swing about the eccentric, being held against the floor-beam by a spring.

When the bridge is nearly closed this hook snaps over a suitable loop on the abutment and is then tightened up by revolving the eccentric by means of a lever and a rope extending to the place where the draw tender stands.



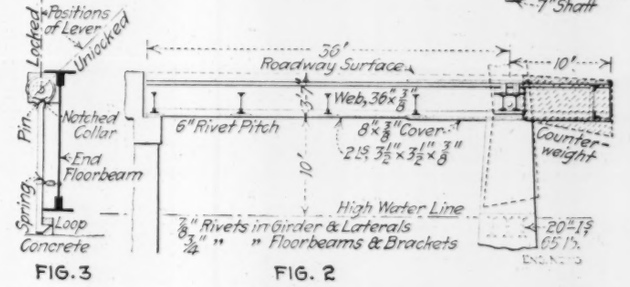


FIG. 2. PLAN, ELEVATION AND CROSS-SECTION OF SIMPLE BASCULE DRAW SPAN

FIG. 3. LOCKING MECHANISM OF DRAW SPAN

The action of this eccentric is to raise the hook drawing the ends of the bridge tightly down on to the wall plates. When it is desired to open the bridge, the rope is pulled and the lever moves, lowering the eccentric and relaxing the tension on the hook. Just before the extreme motion of the lever is reached, the notched collar strikes a pin on the hook and moves it out clear of the loop. The bridge is then free to open.

The cost of the structure as given below does not in-

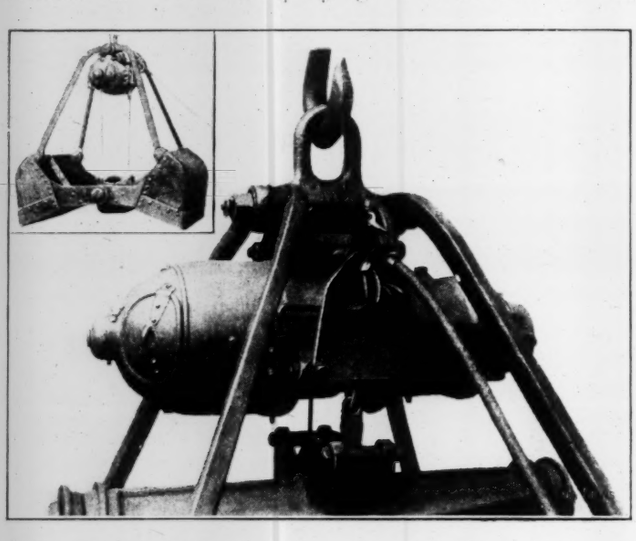
clude the piers. The itemized cost to the contractor of the drawspan is as follows:

Steelwork and machinery, f.o.b. Boston	$945
Freight, erection and painting	625
Wooden floor	200
Concrete counterweight	225
Total	$1,995

or practically $2000 for the drawspan complete and ready for service.

A Motor-Operated Clam-Shell Bucket

A novelty in clam-shell buckets for use in ore and coal handling or for any material handled above water is the use of a separate electric motor mounted on the bucket, which opens and closes the bucket independently of the hoisting apparatus to which the bucket is attached. It is, therefore, of particular value with a variety of hoisting machinery where only one drum is provided. The new electric-motor operated bucket is a complete unit and may be attached to any derrick or crane by a mere hook connection. The accompanying illustrations show the



ELECTRIC-MOTOR OPERATED CLAM-SHELL BUCKET

bucket and operating motor, and also show how easily it is attached or detached to a crane hook.

The blades, blade arms, connecting rods, in fact, all parts of the bucket are standard with the exception of the closing power and the upper casting, which is a solid casting with an eye made to receive the hook of the hoisting block attached to the operating machine.

The motor with the drum, disk clutch and automatic stop are all fastened to the upper head of the bucket by means of two bolts, and are inclosed in a casing which protects the motor from the weather and dust. The motor, electric brake, gearing, disk clutch and winding drum are all mounted on a central shaft. Hand holes are provided at one end for easy access to the working parts of the motor.

By removing the two bolts which hold the casing to the head of the bucket, it may be entirely detached; disconnecting four other bolts divides the shell or casing in two parts which leaves the working parts entirely exposed.

On the shaft which passes through the blade arms is located a sheave incased in a casting, the latter performing the double duty of protecting the sheave and acting as a counterweight to open the bucket. The closing is accomplished by means of a two-part wire-rope line or chain, one end fastened to the motor casing and then passed down over the sheave and back to the winding drum.

The opening and closing is done entirely by the motor, which is usually operated by the crane man, although the controller may be placed in any convenient location. It is not necessary to have it on the operating machine itself; a feature which in some cases has proved of great advantage.

The motor of the bucket shown in the illustration is of 2 hp., direct-current, but motors of the alternating current type may be used. The motor is controlled by a rheostat arranged for two to four speeds each way. The current is conducted to the bucket through a small waterproofed cable, the rheostat end of which is wound on an automatic reel, actuated by springs, which will coil up 500 ft. of cable. Means for varying the tension on the springs to get the desired pull on the cable is accomplished by a small ratchet. The usual tension for drums carrying up to 150 or 200 ft. of cable is approximately 35 lb. For longer distances it may be increased and for shorter distances decreased.

To open the bucket the lever of the rheostatic controller is thrown one way, and to close the bucket the current is reversed. The bucket cannot be overloaded when closing, as the friction clutch operating the drum is so adjusted that it slips when the bucket is entirely closed, or when it grasps some object too large for it to handle. In the opening operation when sufficient line has been wound out to open the bucket to its full extent, an automatic stop prevents the opening and closing line from rewinding on the drum; in other words, the motor rotates but the drum stands still when the bucket is wide open, no matter what position the lever on the controller may be in. This stop is positive in action, the dog or latch being thrown into engagement by the opening and closing line when the bucket has opened to its maximum spread.

The plug connecting the electric cable to the bucket is shown in the illustration. Its location is such that it cannot easily come in contact with anything that would be liable to dislodge it; and the strain on the cable is taken entirely by a snap hook which prevents the possibility of breaking the circuit through an excess of strain on the cable.

The bucket described above is made by the Hayward Co., New York City.

Rapid Trestle Rebuilding—A timber trestle 3500 ft. long and varying from 30 to 40 ft. in height across Newark Bay, in New Jersey, was rebuilt in eleven days and six hours by the Pennsylvania R.R. maintenance-of-way forces, and local contractors. Fire destroyed the trestle early in the morning of June 15. With the burning structure fell 35 freight cars loaded with produce. The removal of the steel rails, wrecked cars and other débris, required about 36 hours, and was hastened by the use of the oxyacetylene blow-pipe, which is a part of the regular equipment of Pennsylvania wrecking trains. The burned trestle piles were sawed off below low-water mark, and capped. Upon this foundation, a new two-track trestle was erected to replace the old four-track trestle, two tracks of which were owned by the Pennsylvania and two by the Lehigh Valley. The new trestle is now operated as a double-track trestle for the use of the two companies jointly. In rebuilding, 1000 men were employed at times and the equipment consisted of fourteen pile drivers, six marine derricks, two marine wreckers, two air compressors and an erection car. The work was done under the direction of C. I. Leiber, division engineer.

Derrick Cars for Bridge Erection

In a paper on the construction of the Dumbarton Bridge of the Central California Ry. (crossing an arm of San Francisco Bay), by E. J. Schneider,* which was presented at a meeting of the American Society of Civil Engineers in March, reference was made to the equipment used in erection. The unloading of material was done by a derrick having the leg of the A-frame pivoted at the base, so that the frame could be swung back and lowered for transportation. For the erection of the draw-span (which was built on the protection pier as falsework), the material was handled by a double-derrick traveler running on four lines of rails on the falsework. At each end of the deck was a single boom and A-frame, the latter braced by timbers leading to each side of the center of the deck. The fixed spans were built on falsework near the shore and floated into position.

In the April number of the "Proceedings" of the above society there is a discussion by L. D. Rights† in regard to this erection equipment, which we have abstracted as follows:

Derrick cars with hinged A-frames have become common in the East, and almost every bridge company owns one or more. Hinged A-frames of wood have been in use for some years, and about ten years ago S. P. Mitchell‡ conceived the idea of building a special steel car with a hinged steel A-frame. A car of this kind was used successfully in connection with the erection of the Warehouse Point Bridge carrying the New York, New Haven & Hartford R.R. across the Connecticut River, below Springfield, Mass.

Mr. Mitchell then substituted heavy steel struts for back-stays, patented the invention, and a number of cars have been built. The A-frame is 23 ft. high, making the total height of the car when in use about 27 ft. Such height of mast is necessary for booms of 50 and 60 ft., and cars have been built recently with masts some 7 ft. higher.

The standard overhead clearance for Middle Western roads is 22 ft. above the rail, and as this is not sufficient to clear the top of the A-frame, Mr. Mitchell has made arrangements to lean the frame back and shorten the back-struts by a series of pin holes in the latter. On some of the Eastern railroads, especially the New England roads, the overhead clearance is very much less than 22 ft.; in some cases it is only about 15 ft., with a minimum of 14 ft. 6 in. This is not much more than sufficient to pass a high furniture car, and it is not always practicable to lean the A-frame back far enough to be able to work under such clearances. Some of the Eastern concerns have attempted to overcome this difficulty by building a heavy steel frame, the top chords of which occupy about the same position as the eaves of a box car. One company has made the front posts of this frame hollow, so that an A-frame can telescope into them. An arrangement of pin-holes enables work to be done with an A-frame at a height very little above that of a box car, or the frame may be raised so as to give a mast of about 18 ft. Several other companies have used this same type, but have hinged a short A-frame on the top of the steel truss, instead of at the car floor.

In regard to the double-end traveler, a single boom for the erection of a truss was not considered good practice several years ago, owing to the number of pieces which had to be held in place while a connection was being made, but the introduction of riveted joints has produced quite a change in methods. Recently, derrick cars have been used very successfully in the erection of riveted trusses, although there are engineers and erectors who maintain that other tools are more efficient.

It is sometimes difficult to decide as to the most efficient method of erecting steel work. Several years ago, the New York, New Haven & Hartford R.R., in extending its six-track line into New York, built a number of overhead bridges for the separation of street and railway grades. The bridges were designed for New York City paving standards and very heavy live loads, and consisted of three trusses with double roadways and sidewalks. The spans were not very long, 120 ft. probably being the average. During a period of about 18 months, four different bridge companies secured contracts for four bridges, the details of which were very similar. They were all overhead highway bridges, with the erection conditions almost exactly the same, and the interesting feature was that each of the four companies used a different method of erection.

One company used a steel gallows, or gantry traveler, reaching above the trusses, and erected the steel from fixed hoists.

The second company erected two heavy steel derricks in the street, one at each abutment, with booms long enough to meet at the middle of the span.

The third company had two derrick cars, one of which was placed on the upper falsework, and was used for erecting the trusses; the other was used down on the railway grade to supply the upper one with material.

The fourth concern built a two-boom mule traveler; that is, a low-decked traveler with two masts and booms. The booms reached out over the falsework to get material from the cars below.

The writer was interested in the erection of one lot of bridges, and watched carefully to see what the other companies were doing, in order to learn whether there was any marked difference in the methods. As far as he could see, each concern was guided in the choice of erection tools by what it had on hand, and there did not seem to be any very great advantage of one kind of tool over the other. He is satisfied that the method in which he was interested was the best; but the representatives of the other concerns feel the same way about their methods, so that the question is still open.

*Contracting Manager, U. S. Steel Products Co. (Bridge and Structural Dept.), San Francisco, Calif.

†Contracting Manager, Lewis F. Shoemaker & Co., New York, N. Y.

‡President, Seaboard Construction Co., Philadelphia, Penn.

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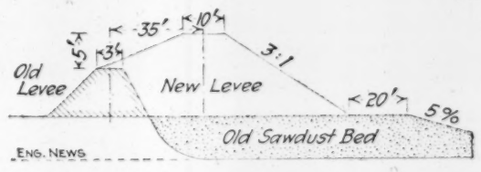
Building a Levee across an Old Sawdust Bed

A levee now being enlarged at Baton Rouge, La., on the Mississippi River, crosses the site of an old sawmill, and the old levee rests on a bed of bark, sawdust and rotting cypress logs, from 6 to 18 ft. deep. This allows the water from the river to seep through. In the new work, in order to prevent this seepage, it has been found necessary before excavating the material for the levee



CROSS-SECTION OF LEVEE ENLARGEMENT AT BATON ROUGE, LA.

(The hatched area represents a sawdust bed that had to be excavated to provide against seepage.)

to strip 6 ft. of this loose material from the toe of the old levee to the river. This is shown in the shaded portion in the accompanying cut. The digging through this material was very hard, mainly on account of the character of the fill which contained logs as large as 3 ft. in diameter and 25 ft. long.

After this material had been excavated and thrown over into the borrow-pit previously excavated, the good material lying underneath was dug and placed in the levee. Although the drag-bucket excavators were occupied 50% of the time in stripping this surface deposit, they averaged 2000 cu.yd. of dirt a day. The day was divided into two shifts of 10 hours each. The work was done with two drag-bucket excavators having 100-ft. booms and 3½-yd. buckets, and the cost is said to be 6 to 7c. per cu.yd. The above information is taken from the May number of the *Excavating Engineer*.

Design of Standard Reinforced-Concrete Retaining Walls

By H. M. Gibb*

The object of this article is to give a method of designing reinforced-concrete retaining walls which is quick, accurate and simple to apply to all common types and which gives results within about 10% of the rigid analysis; also to show a method of making standard designs in which all moments and shears and stresses on the foundation and reinforced slabs of the wall are given in terms of the height of the fill.

The following is the notation used:

ε = angle made by top of fill with the horizontal;
ϕ = angle of repose of fill;
a = angle which the resultant of all forces on the wall makes with the vertical;
h = distance from underside of base to top of fill (see Figs. 2 and 3);
d = distance from top of base to top of fill;
b = width of base of wall;
x = distance of face of vertical stem of wall from toe;
w = weight of fill in lb. per cu.ft.;
p = weight of equivalent fluid in lb. per cu.ft.;
W = resultant vertical pressure;
P = resultant fluid pressure;
s = weight of surcharge in lb. per sq.ft.;
h' = depth of fill equivalent to s;
l = load per lineal foot on span;
$B.M.$ = bending moment;
l = distance c. to c. of counterforts;

$$C = \left[\cos^2 \varepsilon \frac{\cos \varepsilon - \sqrt{\cos^2 \varepsilon - \cos^2 \phi}}{\cos \varepsilon + \sqrt{\cos^2 \varepsilon - \cos^2 \phi}}\right]$$

The loads on a retaining wall are, first, the vertical loads which include the weight of the wall, and the weight of the fill and the surcharge resting on the heel slab; and second, the horizontal thrust of the fill on the vertical stem of the wall.

The action of the fill on the wall is best realized by considering the fill as a semifluid which exerts a vertical pressure due to its own weight, and a horizontal fluid pressure due to its semifluid action, i.e., if y is the distance below the top of the fill then the vertical intensity of pressure at this point $= wy$ and the horizontal intensity of pressure $= py$ (see Fig. 2).

To compute the value of p, Rankine's general formula for a vertical wall is used (Fig. 1) in which it is shown that $p = wC$. The graph gives values of C for all values of ϕ and ε. values of ϕ given for different materials are taken from the American Civil Engineers Pocket Book. Useful slopes have also been shown.

Fig. 2 gives the basis on which the analysis of the wall is made, the weight of the wall and the backfill combined is taken as being represented by the shaded area and of the same weight as the fill, i.e., the shaded part of the wall is taken as the same unit weight as the fill and the weight of the unshaded parts of the wall and fill are neglected. The horizontal pressure is that due to a fluid of p lb. per cu.ft.

Fig. 3 gives the case of the surcharged wall, and it is to be noted that h' is not the height of the surcharge but the height of fill which gives the same intensity of loading as the surcharge. In this case $P = p\left(\frac{h^2 - h'^2}{2}\right)$, but $P = \frac{ph^2}{2}$ is used as it only involves an error of 1% when h' is 20% of h, which is a heavy surcharge.

There is also given on Figs. 2 and 3 a load diagram of pressures on the base; the pressure diagram of the loads on the foundation is the whole of the vertically-lined area, the whole of the horizontally lined area is the load on the heel slab, the cross-lined area represents balanced pressures, and the remaining vertically and horizontally lined areas represent the loads to be resisted by the toe and heel slabs respectively.

The friction of the fill on the back of the wall, and the vertical component of E are neglected.

The conditions for stability are:

1. The resultant must cut the base within the middle third.

2. The resultant must not make an angle with the normal to the base of the wall greater than the angle of friction.

By taking moments about the outside edge of the middle third and equating them to 0, we will get a wall which satisfies the first condition, and on which the maximum toe pressure is twice the average, and the minimum heel pressure is zero.

There is a certain position of the vertical slab on the base slab which will give the minimum length of base, this occurs when $x = \frac{b}{3}$ but as it is not always possible or advisable to build every wall to this proportion, the results of the analysis of Types A to E (the position of wall on base being varied) are given on Fig. 4.

The analysis of Type C is given here in full, the remaining analyses are similar.

$$W = \frac{2\,whb}{3} \qquad P = \frac{ph^2}{2}$$

Taking moments round outside edge of middle third

$$\frac{2\,whb^2}{9} = \frac{ph^3}{6}$$

from which

$$b = 0.87\,h\sqrt{\frac{p}{w}}$$

substituting this value of b, $W = 0.58\,wh^2\sqrt{\frac{p}{w}}$.

Average unit pressure on base $= \frac{W}{b} = 0.66\,wh$.

Maximum pressure on toe $= 1.33\,wh$.

Minimum pressure on heel $= 0$.

Resistance to overturning is obtained by taking moments around the toe, then

$$\frac{Negative\ moments}{Positive\ moments} = \frac{\left(0.58\,wh^2\sqrt{\frac{p}{w}}\right) \times \left(0.58\,h\sqrt{\frac{p}{w}}\right)}{\frac{ph^3}{6}}$$

$$= 2.00$$

To find if wall is safe against sliding

$$Tan\ a = \frac{P}{W} = 0.86\sqrt{\frac{p}{w}}$$

*Engineer of Structures, Dept. of Natural Resources, Canadian Pacific Ry. Co., Brooks, Alta.

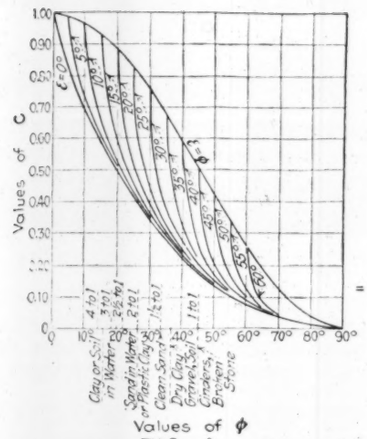
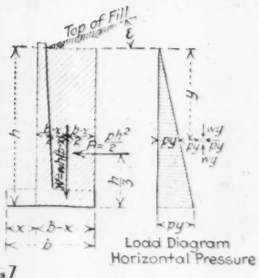
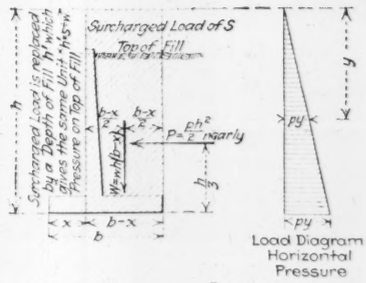


FIG. 1




Surcharged Load of S
Top of Fill
Surcharged Load is replaced by a Depth of Fill h' which gives the same Unit Pressure on Top of Fill
Load Diagram
Horizontal Pressure

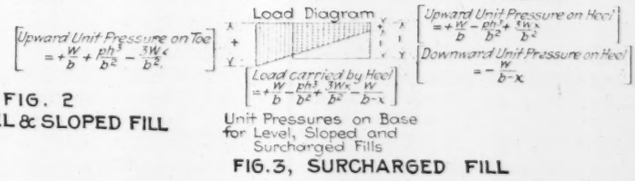

Upward Unit Pressure on Toe
Load Diagram
Load carried by Heel
Upward Unit Pressure on Heel
Downward Unit Pressure on Heel

FIG. 2
LEVEL & SLOPED FILL

Unit Pressures on Base for Level, Sloped and Surcharged Fills

FIG. 3, SURCHARGED FILL

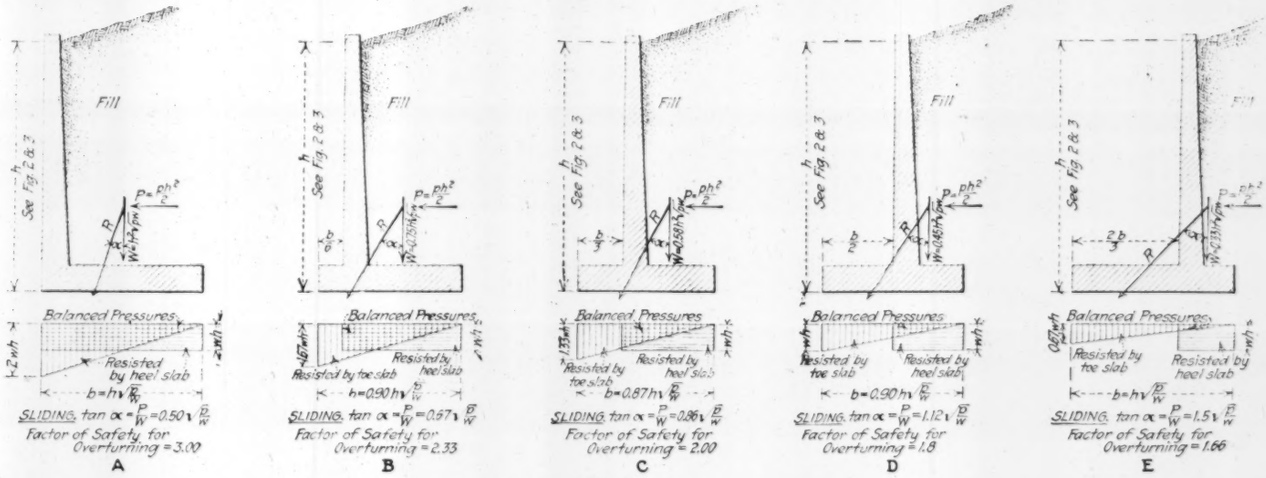

Fill
See Fig 2 & 3
Balanced Pressures
Resisted by heel slab
Resisted by toe slab
SLIDING, tan α = P/W = 0.50 √(P/W)
Factor of Safety for Overturning = 3.00
A
SLIDING, tan α = P/W = 0.67 √(P/W)
Factor of Safety for Overturning = 2.33
B
SLIDING, tan α = P/W = 0.86 √(P/W)
Factor of Safety for Overturning = 2.00
C
SLIDING, tan α = P/W = 1.12 √(P/W)
Factor of Safety for Overturning = 1.8
D
SLIDING, tan α = P/W = 1.5 √(P/W)
Factor of Safety for Overturning = 1.66
E

FIG. 4

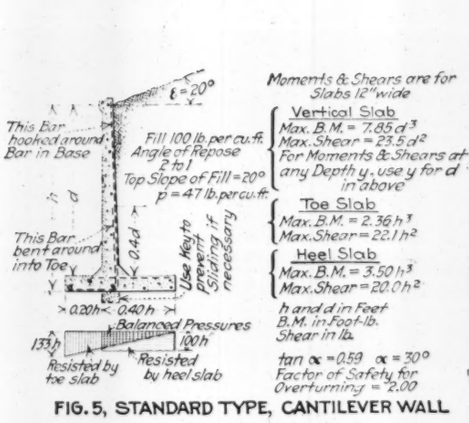
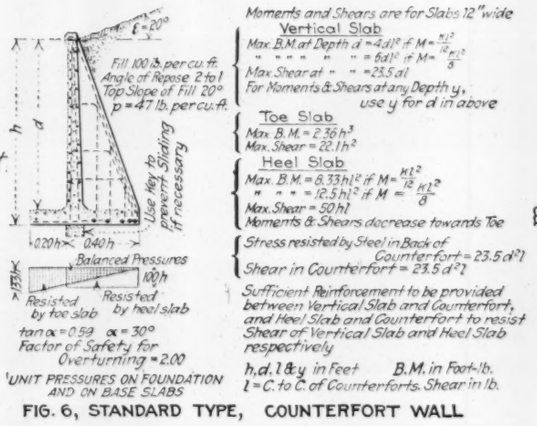

ε = 20°
This Bar hooked around Bar in Base
Fill 100 lb. per cu. ft.
Angle of Repose 2 to 1
Top Slope of Fill = 20°
p = 47 lb. per cu. ft.
This Bar bent around into Toe
Use Key to prevent Sliding if necessary
Balanced Pressures
Resisted by toe slab
Resisted by heel slab
Moments & Shears are for Slabs 12" wide
Vertical Slab
Max. B.M. = 7.85 d³
Max. Shear = 23.5 d²
For Moments & Shears at any Depth y, use y for d in above
Toe Slab
Max. B.M. = 2.36 h³
Max. Shear = 22.1 h²
Heel Slab
Max. B.M. = 3.50 h³
Max. Shear = 20.0 h²
h and d in Feet
B.M. in Foot-lb.
Shear in lb.
tan α = 0.59 α = 30°
Factor of Safety for Overturning = 2.00

FIG. 5, STANDARD TYPE, CANTILEVER WALL


ε = 20°
Fill 100 lb. per cu. ft.
Angle of Repose 2 to 1
Top Slope of Fill 20°
p = 47 lb. per cu. ft.
Use Key to prevent Sliding if necessary
Balanced Pressures
Resisted by toe slab
Resisted by heel slab
tan α = 0.59 α = 30°
Factor of Safety for Overturning = 2.00
Moments and Shears are for Slabs 12" wide
Vertical Slab
Max. B.M at Depth d = 4 d l² if M = Kl²
" " " " " = 6 d l² if M = 1/2 Kl²
Max. Shear at " " = 23.5 d l
For Moments & Shears at any Depth y, use y for d in above
Toe Slab
Max. B.M. = 2.36 h³
Max. Shear = 22.1 h²
Heel Slab
Max. B.M. = 6.33 h l² if M = Kl²
" " " = 12.5 h l² if M = 1/2 Kl²
Max. Shear = 50 h l
Moments & Shears decrease towards Toe
Stress resisted by Steel in Back of Counterfort = 23.5 d² l
Shear in Counterfort = 23.5 d² l
Sufficient Reinforcement to be provided between Vertical Slab and Counterfort, and Heel Slab and Counterfort to resist Shear of Vertical Slab and Heel Slab respectively
h, d, l & y in Feet B.M. in Foot-lb.
l = C. to C. of Counterforts. Shear in lb.

'UNIT PRESSURES ON FOUNDATION AND ON BASE SLABS

FIG. 6, STANDARD TYPE, COUNTERFORT WALL

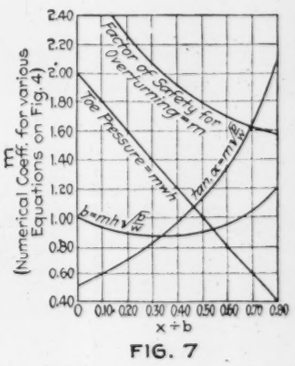


FIG. 7

A Shortened Method for the Design of Reinforced-Concrete Retaining Walls

(Devised by H. M. Gibb.)

On Fig. 4 five types (*A*, *B*, *C*, *D*, *E*) of retaining walls are given with the magnitude of the unit pressures, ratio of width of base to height of fill, resistance to overturning and values of tan α.

The following principles are noted:

1. The length of the base varies as $\sqrt{\frac{p}{w}}$.
2. As the vertical stem is moved back from the toe toward the heel, the unit toe pressures decrease in intensity, the length of base b decreases till it is a minimum when the face of the stem is $\frac{b}{3}$ from the toe, after this it increases in value, but between positions of $\frac{b}{6}$ and $\frac{b}{2}$ from toe, the difference amounts to only 3%.
3. The resistance to sliding decreases as the vertical stem is moved back from the toe toward the heel.
4. The resistance to overturning decreases as the stem is moved back from the toe toward the heel.

If the above wall Type *C* is wanted with a factor of safety against overturning of 3, then by taking moments round the toe and equating the moment of W to three

times the moment of P we will get $b = 1.05\,h\sqrt{\frac{p}{w}}$.

From equations on Figs. 2 and 3 the unit pressures on the base at toe and heel will be found to be $+0.92\ wh$ and $+0.42\ wh$, respectively.

If it is required in Type C to find the length of base necessary to prevent sliding, then

$$Tan\ a = \frac{P}{W} = \frac{\frac{ph^2}{2}}{\frac{2}{3}\,whb}$$

and

$$b = \frac{0.75\,ph}{w\ tan\ a}$$

After substituting the values of p, w and tan a in this equation, the solution of the pressures on the base can be made by the equations on Figs. 2 and 3.

Design of Standards

Where a large number of walls retaining similar fill but of varying height are to be designed, the standardization can be carried further than shown on Fig. 4, and in Figs. 5 and 6 are given standard cantilever and counterfort walls based on Type C.

The fill designed for weighs 100 lb. per cu.ft., and slopes on top at an angle of 20°, and has an angle of repose of 2 to 1. From Fig. 1, $C = 0.17$ and $p = 47$ lb. per cu.ft.

In Figs. 5 and 6 are shown a cantilever and a counterfort wall with all moments and shears and unit pressures in toe, heel and vertical slabs expressed in terms of the height of the fill, these values are arrived at by substituting the values of p and w in Type C, Fig. 4, and calculating the moments and shears by the usual methods.

To find the stress in the back of the counterfort the lever arm of the beam is for this type $\frac{d}{3}$ approximately. Then

$$Stress = \frac{pd^3l}{b} \div \frac{d}{3} = 23.5\ d^2l$$

The value of tan a is 0.59, and if this is too high for the material on which the wall is founded, it will be necessary to introduce a key in the base, as shown in dots, to give greater resistance to sliding.

Fig. 7 is a graph of the numerical coefficients in equations on Fig. 4 for different values of $\frac{c}{b}$ and by means of this graph all intermediate values of these coefficients can be arrived at.

In conclusion it may be stated that in the cantilever wall, all three slabs (vertical, heel and toe slabs) of the wall are cantilevers, with the support at their junction; the toe slab is reinforced in the bottom, the heel slab in the top, and the vertical slab in the back.

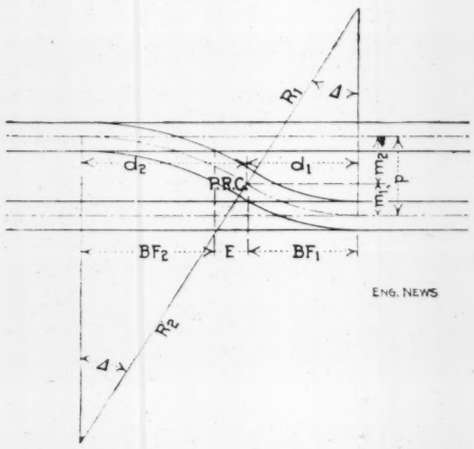
In the counterfort wall, the vertical slab and the heel slab are beams spanning between the counterforts, which are their supports, the vertical slab is reinforced in the front, and the heel slab on the bottom, and with short lengths of bar over the counterforts to take negative moments; the toe slab is a cantilever, reinforced on the bottom.

In all cantilever slabs special attention should be taken to insure that bars are given sufficient bond over the supports to develop their full strength.

NOTES

A Crossover Problem—A somewhat unusual track problem is solved by H. V. Knouse (Civil Engineer, Colorado-Yule Marble Co.), Marble, Colo., as follows:

REVERSE-CURVE CROSSOVER WITH UNEQUAL FROGS—Given the frog numbers n_1 and n_2 and the corresponding radii R_1 and R_2 of the turnout curves, and the distance between track centers p. To find the distance between frog points E, and the point of reverse, P. R. C. The turnout curves are continued to tangency at the P. R. C.



Reverse-Curve Crossover between Straight Tracks with Unequal Frogs

The two central angles Δ being equal,

$$\text{vers}\ \Delta = \frac{m_1}{R_1} = \frac{m_2}{R_2}$$

or

$$\frac{m_2}{R_2} = \frac{p - m_2}{R_1}$$

or

$$m_2 = \frac{R_2}{R_1 + R_2}\,p$$

$$\therefore \text{vers}\ \Delta = \frac{p}{R_1 + R_2}$$

This gives the angle to the P. R. C. To get the distance between frogs,

$$E = d_1 + d_2 - (BF_1 + BF_2)$$

The switch leads BF may be computed or found from tables, and

$$d_1 = R_1 \sin \Delta$$

$$d_2 = R_2 \sin \Delta$$

When the frogs are equal, $R_1 = R_2$, and

$$\text{vers}\ \Delta = \frac{p}{2R}$$

which corresponds to the formula given in Searles "Field Engineering," p. 173.

It should be noted that there is a limit to the diversity of frogs that may be used, depending on the distance between track centers. The maximum difference is determined by the proportion

$$\frac{\text{gage}}{p - \text{gage}} = \frac{R_1}{R_2}$$

Weighing the Plot as a Means of Measuring Land Areas—Louis C. Mathey, Spot, Tennessee, writes us that he has for 34 years been obtaining land areas by the simple process of accurately plotting the survey to scale on good paper, then cutting away the paper outside of the boundary with a sharp knife and weighing the folded plot of the tract on a chemist's or jeweler's scale accurate to $^1/_{01}$ grain. He sends us a sample map of a tract of 16.651 acres, plotted on a scale of 80 rods to the inch, on a sheet of 18x24-in. paper, which weighed 42 $^8/_{32}$ grains. While the plan might serve as a rough check on computations, the limit of error in plotting and trimming, as well as in weighing, would make the scheme only applicable where land is of very low value.

The Solution of a Special Problem in Reverse Curves, published in our Field and Office columns of June 26, p. 1322, is criticized by William Vetter, 2463 Marion Ave., New York City, who writes us as follows:

I have read with interest the article in the June 26 issue on the analysis of reversed curves. I would like to call the writer's attention to the treatment of compound curves given in Searle's "Field Engineering." All the formulas for compound curves may be easily adapted to apply to reversed curves. Since the centers are on opposite sides of the curve, the central angles Δ_1 and Δ_2 have different signs. Hence the two tangents S_1 and S_2 also have different signs.

By making the proper changes of signs in formulas (100), (101) and (102) we have:

$$\text{Cot } \tfrac{1}{2}\gamma = \frac{S_2 - S_1}{S_2 + S_1} \times \cot \tfrac{1}{2}\Delta \qquad (100)$$

$$\Delta_2 - \Delta_1 = \Delta \qquad (101)$$

$$R_1 = \tfrac{1}{2}(S_1 + S_2)(\cot \tfrac{1}{2}\gamma + \cot \tfrac{1}{2}\Delta_1)$$
$$R_2 = \tfrac{1}{2}(S_1 + S_2)(\cot \tfrac{1}{2}\gamma - \cot \tfrac{1}{2}\Delta_2) \qquad (102)$$

The other formulas may be derived from these in the same manner as treated in above mentioned textbook.

I would like to call attention to the fact that the line of centers of the reversed curve is tangent to a fixed circle, which fact is not apparent from an inspection of the sketches.

Bending 10-in. Cast-Iron Pipe—A part of the pipe line built by the United Fruit Co. for conveying water from the Guayabo River to the town of Preston, Cuba, lies in a cañon, which for about two miles from the dam is crooked, making it impossible to use such easy curves as could be made in the pipe joints. Owing to an oversight no bends or sleeves were ordered with this pipe; and as it might have taken a month or more to send and get specials, the local engineer decided to bend some of the straight pipes. This had been done frequently with steel or wrought-iron pipes for the sugar mill, and they followed the same course of procedure with the cast-iron pipe, with entire success, as they did not break or spoil a single pipe. The pipes were bent to various radii, the shortest being 50 ft.

A cradle of old rails was first constructed with the desired amount of curvature. About 1 ft. at each end of the pipe was left outside the fire, to prevent collapse of the pipe, and a fire of hard wood was built under and around the remainder of the pipes. Six or eight pipes were bent at a time. In from 1½ to 2 hours after starting the fire a length of pipe was hot enough to bend, and settled from its own weight to the cradle prepared to receive it.

The pipe was 10 in. in diameter, $\frac{9}{16}$ in. thick, corresponding to class D of the New England Water Works Association Specifications, the weight of which is 760 lb. per 12-ft. length. [Abstract of a paper read by Charles W. Sherman, of Metcalf & Eddy, Consulting Engineers, Boston, Mass., before the New England Water Works Association, Feb. 12, 1913.]

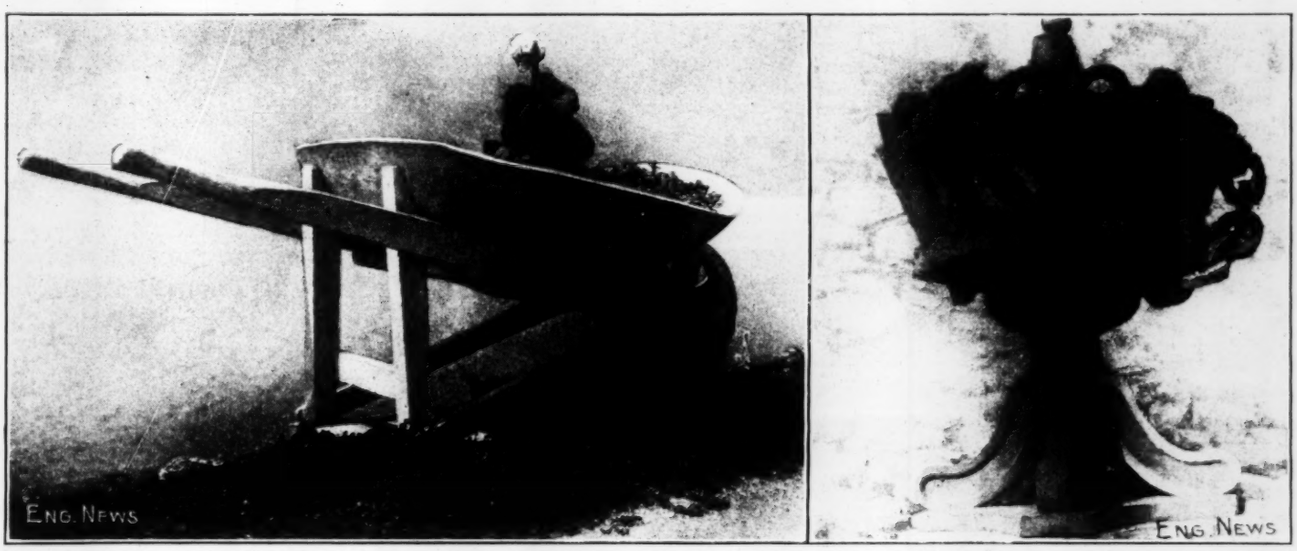
Field Glasses for Track Inspection are being used on the Prussian State Railway by maintenance-of-way engineers and inspectors, according to an article in the "Railway Engineer," of London. It is stated that in a field glass for the detection of rail defects, one of the first requirements to be fulfilled by the special binocular is a large field of view, since it is only possible to detect such defects by comparing single rails over a long distance. This is especially the case when crossings and curves are under observation. The critical observation of short lengths of rails is only of relative value.

It is advisable when using the glasses for inspection to face the sun so that the rails may appear as bright as possible, which will render any defects quite apparent. Rails which are contorted, uneven in height or gage, have loose or low joints, faulty curves, and other defects can then be easily recognized at long distances. This refers, of course, to track defects rather than defects in the quality of the rails, but mirrors used on the Pennsylvania R.R. for detecting flaws in rails were described in "Engineering News," Oct. 24, 1912.

The special field glass mentioned as being used in Germany is known as the "Telact" and is manufactured by Carl Zeiss, of Jena. It has a field view of 345 ft. at a distance of 3000 ft. with a magnification of 8 times.

Decorative Concrete—The accompanying illustrations are interesting as showing what can be done with concrete by the use of a little ingenuity. The implements used were ordinary wooden molds, in some instances fireclay molds, and the artist was an ordinary workman, employed as a caretaker at the water-works plant of the Bethlehem City Water Co., South Bethlehem, Penn. The objects shown and various others to the number of about forty, including some statuary, were made at odd moments for amusement.

The concrete chain illustrated was cast a link at a time, and the links are reinforced with wire. Reinforcement is, of course, necessary in parts of the other objects.



DECORATIVE CONCRETE OBJECTS MADE BY A WATER-WORKS EMPLOYEE FOR AMUSEMENT

With due apologies to the water company, we must confess the general appearance of the water-works grounds bears some resemblance to a graveyard.

A New Method of Testing Sand was described by W. B. Reinke before the recent meeting of the American Society for Testing Materials in Atlantic City. The test is made in order to more closely approximate actual conditions in the field, and to this end small stone are introduced into the mixture, afterward to be removed, and an excess over the so called normal amount of water is used.

Mr. Reinke describes the test as follows:

The sand to be tested and Ottawa sand are first made into mortars of normal consistency, using preferably cement from the work. These mortars are then made into briquettes, the whole operation being in accordance with the prescribed method of this Society for the making of sand briquettes when testing cement. The amount of water necessary to make a mortar of normal consistency having been found, a new batch of sand and cement of the desired proportions is thoroughly mixed dry and placed in a cylinder that can be revolved together with a number of pebbles, so that the proportion of cement, sand and stone will approximate that desired in the actual work. To this mix is added enough water (generally about 50% more than needed for normal consistency) to make a mortar having work consistency. The cylinder is then closed and revolved for five minutes. The pebbles are picked out by hand and the mortar is made into briquettes. This test more closely approximates the conditions found in actual work and the strength obtained from briquettes made from such a mortar, though, as a rule, far below those obtained from a mortar of normal consistency, correspond to those that may reasonably be expected from the mortar if the sand and the same cement are used in construction.

Pumping Out Foundation Pits for column footings or pedestals so as to remove any water before depositing concrete, is very desirable if good, sound concrete is to be obtained. This is frequently neglected, however, and thin, slushy concrete is dumped into the water, with the result that there is a great excess of moisture, and the proper setting of the concrete may be retarded. An interesting method has been used in some building construction at Chicago to remove the relatively small amount of water in the pits for spread foundations of footings in the clay soil. A suction hose attached to a small electrically-operated trench pump had the end fitted with a cast-iron cylinder on which were formed four nipples for attaching small hose lines. The end of the cylinder was plugged. Four pits could be drained simultaneously, and just before depositing concrete in any pit a man would take one of the hose pipes and move the end about in the pit so as to take the water from any low spots.

The Internal Temperature Range in Concrete Arch Rings has been made the subject of investigation at the Iowa State College of Agriculture and Mechanic Arts, and has been reported on in a bulletin of that college by Messrs C. S. Nichols and C. B. McCullough. The studies were made on two bridges in central Iowa, one over a period of two years and one over one year. Thermometers were imbedded in the masonry at various distances from the surface and read almost daily throughout the periods noted. The main useful conclusion made by the authors is that in the climate of Iowa, with a mean daily variation through the year of about 102° F. (and, according to the reports of the U. S. Weather Bureau, a maximum daily range for the year of 153° F., this range being somewhat abnormal and not during the period when the investigations were made), the average internal range of temperature in a reinforced-concrete arch rib would be 80° F. For such an average range, the variation of internal temperature with depth from surface is expressed by the formula

$$y = 90^\circ - \frac{x}{}$$

where

y = Range of internal temperature in degrees F.;
x = Distance of point from surface in inches.

❖

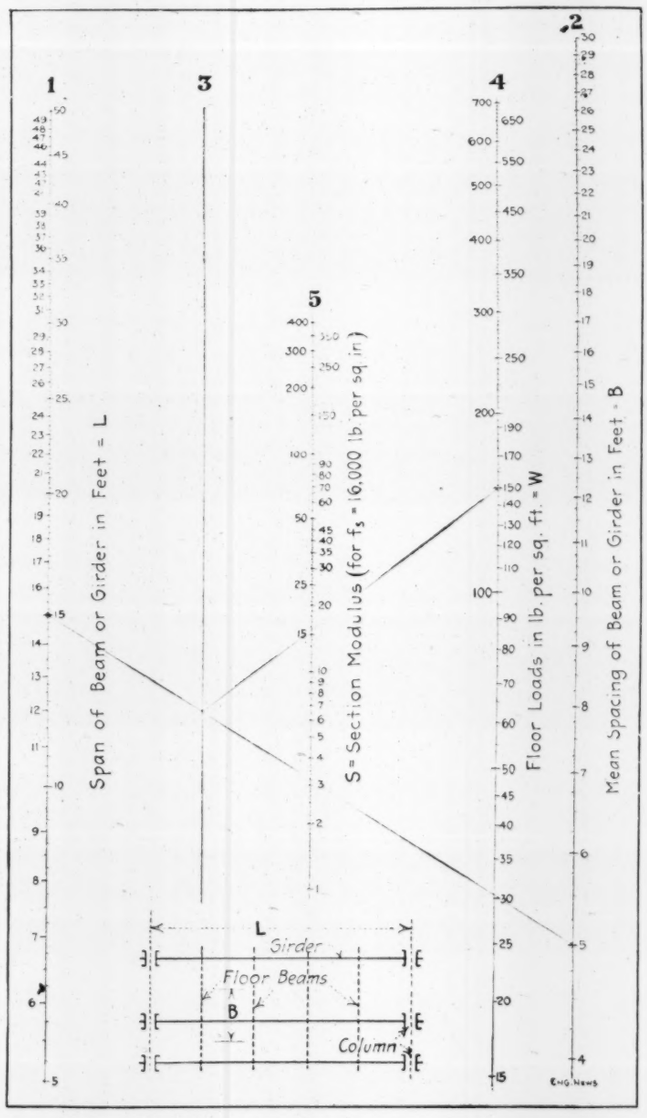
A Chart for Determining the Section Moduli of Floor Beams

The accompanying diagram has been devised by Joseph Di Stasio, Fagan Iron Works, N. Y. City, to determine the section modulus of a beam or girder in a regular floor framing, similar to that shown at the bottom of the cut. The diagram is of the type known as an "alignment chart" and has been designed according to the methods outlined in Prof. J. B. Peddle's "The Construction of Graphical Charts."

L is the span of the girder under consideration, B is the mean spacing of the girders and W is the uniform floor load per sq.ft. Then the section modulus of the girder $= S = \frac{12WBL^2}{8 \times 16{,}000}$, assuming 16,000 lb. per sq. in. as a designing unit stress in the steel. The diagram solves this equation as follows:

Connect span of girder as shown on line (1) with mean spacing of beams as shown on (2); where this line intersects (3) connect point with loading per sq.ft. as shown on (4); read section modulus where this last line intersects (5). Thus the accompanying diagram shows the procedure for a span of 15 ft., a spacing of 5 ft. and a loading of 150 lb. per sq.ft.; the section modulus being 15.8.

The above assumes a uniform loading. When the girder under consideration is subjected to evenly spaced concentrated loads such as is shown in the example in the cut, the section modulus as found on the diagram is subject to the corrections given in the following table.



TO FIND SECTION MODULUS OF GIRDER

Connect span on (1) by a straight line with spacing on (2) and then connect intersection on (3) with floor load on (4). The intersection on (5) gives the required Section Modulus.

[Devised by Joseph Di Stasio, N. Y. City]

It will be noted that no correction need be applied when the framing beams are odd in number.

TABLE OF COEFFICIENTS

No. of floor beams symmetrically framing into girder n	Multiply sect. mod. by coeff. given below c
Any odd number	1.000
4	0.889
4	0.960
6	0.980
8	0.988
10	0.992
12	0.994
14	0.996
16	0.997
18	0.997
20	0.999

Let n = any even no. of beams.

Coeff. $= c = \frac{n\,(n+2)}{(n+1)^2}$

For any unit stress other than 16,000 lb. per sq.in., the proper section modulus is in the same proportion to that found on the diagram as is the desired unit stress to 16,000.

The chart may also be used to obtain the mean spacing of floor beams: i.e., knowing the span of any bay, the floor loading and size of beam, the spacing is obtained by working the chart in reverse order to that noted above.

EDITORIALS

The New Law for Railway Arbitration

A good illustration of the celerity with which Congress can enact legislation in the face of a sufficiently great emergency, is the prompt passage of the Newlands bill, amending the Erdman railway arbitration law. On July 14, at a conference held at the White House, an agreement was reached between the presidents of important Eastern railway companies and the heads of the principal railway labor unions, as to the terms of a bill acceptable to both sides. The bill was rushed through both houses of Congress and signed by the President on the following day, July 15.

The bill creates an officer to be known as the Commissioner of Mediation and Conciliation, to be appointed by the President for a term of seven years, who is to serve with two other government officials as a Board of Mediation and Conciliation. The Commissioner is to be independent of the Department of Labor, and will devote his whole time to the work of the office. He and his associates are empowered to offer their services to the parties to any railway labor dispute in the endeavor to bring about an agreement between them.

In case the Board of Mediation is unable itself to adjust the differences, it shall endeavor to secure an agreement of the parties to the dispute to submit their differences to arbitration. The Board of Arbitration may be of either three or six members. In the latter case, the employers and the employees each select two and the four are to name the remaining two within fifteen days after their first meeting. The Act further specifies in detail what the agreement to arbitrate shall contain. It is probable that the provisions of the law will at once be put into effect in creating a board to arbitrate the demands of the conductors and trainmen of Eastern railways for an increase of wages.

A new element was brought into this railway labor controversy on July 16, however, when the Eastern railway managers presented to the heads of the railway labor unions a series of proposed changes in existing rates of pay and in working conditions of conductors and trainmen, the effect of which is to reduce the compensation of certain conductors and trainmen which are claimed to be excessive for the service rendered. For example, the railways propose that all monthly guarantees shall be abolished, and that where certain schedules give opportunity for extensive mileage in a limited number of hours for passenger conductors and trainmen, that the mileage rate should be reduced.

This is the first time that railway companies have ever proposed to submit to arbitration the question whether a cut in wages should be made, and it has taken the employees by surprise. The Union leaders have admitted, however, that the employers have as much right to appeal to an arbitration board on a proposed cut in wages as the employees have in a proposed increase. In fact, when one recalls that in the past the only preliminary to a cut in wages has been a circular from general office headquarters announcing the cut, the proposal of the railways to arbitrate such reductions is tantamount to a recognition of the employees' rights in collective bargaining.

⁂

A System of Medical Inspection for Employees

How to treat in a systematic, humane and businesslike way the cases of illness which occur in any organization having a large operating force, has been solved in an interesting manner by the Brooklyn Rapid Transit (street railway) Co.

For some years past the company has been engaged in so called welfare work for its employees to a considerable extent. An investigation of this work made by Col. T. S. Williams, the President of the company, showed a wide variation on different divisions of the company's lines in the treatment of excuses offered for illness and absence. On some divisions, in fact, it had become a common matter for men to report themselves ill when they wanted to get time off for purposes of recreation.

To meet this and other problems, it was decided to establish a medical-inspection bureau, and the result of six months' working has been so satisfactory that it has been made a permanent part of the organization. The operation of this medical-inspection bureau may be briefly summarized as follows: When a man reports himself sick at his depot or terminal, he is given an excuse card which instructs him to go to the nearest examination office in the next office hour for medical inspection. At the inspection office the doctor examines the man and either excuses him indefinitely, excuses him for a limited period in a case of minor illness, or if he finds that the illness is feigned, orders him to report back to his depot, where he is disciplined. In case a man is excused indefinitely he receives medical treatment during his illness, at home or at the doctor's office, as required, and upon recovery receives a certificate stating the time when he is to report for work.

The results of six months' operation of the system have shown a reduction of 23 per cent. in the number of days lost on account of illness by the operating force of the company as compared with the first six months of 1912. It appears that this reduction in the loss of time through illness is due, not to a decrease in the number of men reporting sick, but in the shortening of the period of illness. Formerly a man contracting a minor ailment would continue to work without treatment until obliged to lay off, and then would be absent for a week or more. Under the present system, the men welcome the opportunity to receive examination and treatment as soon as they feel indisposed, and are often able to return to work in one or two days. It may be explained that the work is carried on by the same organization which conducts the free medical attendance in connection with a mutual benefit association among the employees of the company.

Should the United States Participate in International Congresses?

Is there any good reason why the United States should refuse to take part in conferences with representatives of other nations having to do with matters of common interest? At the recent International Road Congress in London, where more than thirty different nations were represented by delegates officially appointed by their respective governments, the United States took no part. There were, it is true, delegates present there representing the City of New York, the State of New York and the State of Massachusetts, but the Federal Government itself was not represented.

It has long been the custom for the Secretary of State to appoint one or more delegates to such International Congresses to represent this Government, making his selection from experts of high standing in the field covered by the Congress. Such delegates have, as a rule, paid their own expenses so that their appointment has placed no burden whatever upon the Federal Government. This long established practice has now been ended by a rider tacked onto the last Deficiency Appropriation bill which prohibits the Executive from extending or accepting an invitation to any International Congress or sending delegates thereto representing this Government, except by authority of a special act of Congress.

What the motive may have been which placed this particular piece of foolish legislation on the statute book, we have no knowledge. It is clear, however, that the United States is hereafter to go unrepresented in practically all International Congresses held to consider scientific and technical matters. There are very few cases where those who are interested in the success of such a meeting will be able to devote the time and money and energy necessary to secure the passage of a special act of Congress to give us representation.

We believe this to be in the highest degree, unfortunate. It is not that the actual technical work accomplished by these International Congresses is of the highest importance. We are well aware of, and have plainly set forth in these columns at various times, the limitations under which such International Congresses inevitably operate. Viewed from a broad aspect, however, the ends attained by such Congresses are even more important than the technical questions they are called to discuss.

Public sentiment in the United States undoubtedly favors the policy of good fellowship and comradeship with the other nations of the earth. For this country to stand aloof from every such international gathering and refuse to be officially represented, will inevitably cause misunderstanding of our position, and injure our standing as a nation in technical and scientific matters.

This Congressional legislation grows out of the tendency that is nowadays so strong for the Legislative department to usurp authority that properly belongs to the Executive. No one who knows the limitations under which Congress works, can doubt that the Executive Department of the Federal Government is far better able to wisely decide whether delegates should be sent to represent this government at international meetings than is Congress.

There is no doubt that the foolish law ought to be repealed at the earliest possible date. It will probably take some years of agitation by technical and scientific societies, however, before any such repeal can be gotten through Congress, so that the nation may again be properly represented in international gatherings.

The Spacing of Distant Signals

In the recent report of the Interstate Commerce Commission on the collision at Stamford, Conn., on the N. Y., N. H. & H. R.R., one of the recommendations was that a distant signal should be placed at least 2500 ft. back of the home signal, instead of 1800 ft., which was the distance at Stamford.

The question raised by this recommendation is by no means a new one, nor is it one to which there is only one side. The newspapers generally have interpreted this clause in the Interstate Commission's report as implying that the railway company was culpable in not placing the distant signal so far back of the home signal that the fastest train could be stopped by the application of brakes in the distance between the two. Every signal engineer knows, however, the serious objection to placing mechanically operated signals too far away from the signal tower. If the signal is operated mechanically, a greater length of wire increases the liability to trouble from temperature variations. Of course, the farther away the signal is, the more difficult it is for the towerman to make certain that it is in good order and working properly.

It must be borne in mind that the distant signal represents an advanced development of the signaling art. A large proportion of the signaling equipment on American railways is not equipped with distant signals. It will be recognized, of course, that when a distant signal is used it must be so installed that it cannot show safety when the home signal is showing danger, or it may be the cause of serious disaster. So far as possible, this rule is complied with in the installation of distant signals; but the farther away the distant signal is from the home signal, the greater is the possibility that through some mechanical mischance or perhaps by malicious intent, such a wrong signal indication may be given.

The Interstate Commerce Commission bases its assumption that a distant signal ought to be at least 2500 ft. back of the home signal, on the theory that a fast train must be stopped in that distance. It is not at all necessary to assume, however, that the engine runner is not to apply brakes until he reaches the distant signal. In good weather and on straight track he should be able to identity a distant signal at least 1000 ft. before he reaches it and check his train at once if it is at "caution." In fog or storm, when he cannot see the distant signal until he is close upon it, he should reduce the speed of his train as he approaches the distant signal. This is what he does, of course, in approaching a home signal, which has no distant signal in front of it, and the same rule should apply. Viewed in this light, it will be evident that to put the distant signal several hundred feet farther back of the home signal, as the Interstate Commerce Commission recommends, might increase danger to trains instead of reducing it.

Another alternative, of course, is the substitution of automatic electric signals for manually operated signals, and this course has been adopted by the New Haven Railroad. In the automatic system, as is well known, the distant signal which indicates the position of the home

signal is placed on the signal post of the next home signal back. The signal spacing to be adopted on the New Haven main line will bring the distant signal 5000 ft. back of the home signal. It must be recognized, however, that very few railways can afford signaling of this high class. What is most needed for public safety is that some elementary for mof block signaling should be adopted on the large mileage of railways that are still running with no fixed signal protection whatever.

The City of Denver and Denver Union Water Co. Decision

After a long, hard struggle for municipal ownership of water-works, the City of Denver, Colo., seems to be in a position to attain its desired end in its own way. This has been brought about by a decision of the Supreme Court of the United States which holds that the city is compelled neither to buy the works of the Denver Union Water Co., as maintained by the trustees for the bondholders, nor to extend the franchise of the company for twenty years, as claimed by the company itself. The decision also establishes the contention of the city that the company is without any franchise rights in the streets of Denver and leaves the city free to issue bonds for municipal works which were voted several years ago.

The case involves a number of points of special interest. One of these is that although the city and company agreed to an appraisal for purchase and a fixing of rates for water service under a new contract, after which the electorate was to decide between the two alternatives, this agreement was nullified by the fact that the commission of engineers created under the agreement to determine the purchase price and fix new contract rates failed to do the latter. By this failure, and through no fault of either the company or the city, the court states, the agreement just mentioned became of no effect. Another point or series of points of interest centers around city charter provisions which limit all franchises to 20 years and which provide that no franchise to use the streets of Denver shall be "granted except upon the vote of the qualified taxpaying electors" of the city.

The water company and the holders of its bonds now face the alternative of selling to the city on the terms offered by the latter or of meeting the competition of municipal works. Since the bonds issued by the company are due in 1914 and since the voters of Denver have already voted against extending the company's franchise and in favor of issuing bonds to build municipal works, this alternative is a serious one. It is all the more serious because, although the appraisers already named valued the property of the company at $14,400,000, the price offered the company by the city authorities two or three years ago was only $7,000,000.

The decision of the United States Supreme Court is interesting from various legal and governmental viewpoints, but as it seems to establish no new points of law and involves nothing of direct engineering interest except disparities in valuation, we refrain from quoting it or abstracting it at length.*

It is sufficient to say that the court decided the case in accordance with the plain commonsense meaning of the language of the franchise of 1890, of certain limiting portions of the city charter, and of the appraisal and rate framing ordinance of 1907, and rejected in rather sweeping language all pleas of the company that its constitutional rights were infringed by the mooted provisions of any of those documents.

The gist of the interpretation of the franchise of 1890 was that two separate provisions, one declaring that at the end of 20 years the city "*may*" buy the works and the other that at the same time it "*may*" renew the contract for 20 years, in each case provided it *elects* to do so, are not mandatory alternatives, but independent options, and that the city did not elect to exercise either option. Therefore, the franchise lapsed several years ago. Furthermore, the court held that the city had clear authority to build works if the voters decided to do so, and that they had so decided.

Although the position in which this decision places the company is unfortunate, it seems to be in full accordance with legal justice and no worse for the company than many a decision under which franchise companies have scored heavily against cities. The company finds itself without a franchise and liable to competition from a municipal plant unless it sells to the city on such terms as the city may offer. The price previously tendered by the city is less than half the valuation of the appraisers some years ago and less than the outstanding bonds of the company, to say nothing of its capital stock.

It may seem a simple enough matter for the city and the company to come to reasonable terms, but it should be remembered that there has been a long and bitter fight between the two; that the people are determined on municipal ownership; that they believe the price set by the appraisers grossly excessive, and based on unwarranted assumptions. Moreover, the people of Denver, or a majority of them, as shown at various elections, believe that they have been waging a war against an unholy alliance between the public-service corporations, various immoral agencies and corrupt politicians. A legal victory in the highest court of the land, following a series of hard-fought victories at the polls, puts the people of Denver in no temper to compromise with a vanquished foe, as many of them regard the Denver Union Water Co.

It should be remembered also that for many years the people of Denver have jealously guarded their rights to establish municipal water-works and to pass upon every franchise grant or extension at the polls. These rights were guaranteed in their home rule charter, were in force when the franchise of 1890 was granted, and the charter itself guaranteed them still further. The water company seems to have failed to recognize the existence of these guarantees or to have believed that it could override or circumvent them. The people have clung to their rights and have been backed by the Supreme Court of the United States.

Notwithstanding all this we think it will be not only unfortunate but also a great economic waste if some way is not found to prevent a duplication of water-works at

*The case is The City and County of Denver, Petitioners, vs. the New York Trust Co. and Do., vs. The Denver Union Water Co., on Certiorari to the United States Court of Appeals for the Eighth District. It was decided on May 26, 1913, the unanimous opinion of the court being delivered by Judge Van Devanter. The original suit was to prevent a proposed bond issue for municipal works. The lower courts granted a temporary injunction against the bond issue. The Supreme Court reversed the lower courts and ordered the bills dismissed.

Denver—particularly a duplication of the distribution system, since the water now being sold in Denver could probably be readily devoted to some other even though less profitable use. It seems highly probable that some purchase plan will be evolved, now that the legal atmosphere has been cleared.

It is quite possible that considerably less than the $14,400,000 agreed on by the appraisers could be accepted by the company with no real loss other than a loss on watered capitalization. The courts, so far as we know, have never passed on the city's claim that the basis of valuation was unfair. Obviously, the capitalization (stock and bonds) of the company is no standard of value for this any more than for any other public utility company. In this case, we understand, even before the decision, the bonds were considerably below and the stock far below par.

We have said that it would be a pity to duplicate the water-works of Denver by building a municipal plant. We may now add that Denver is one of the few cities of the United States where competitive water-works have already been tried. Rival companies were in existence for a number of years, with cut rates and other interesting but expensive accompaniments. Consolidation followed. This may explain, in part, the discrepancy between the city's idea and the company's idea of the value of the present works.

Certainly it is high time for Denver to bring its long and expensive water-works struggles to an end. Competitive private water-works have been tried, with probable disaster to capital and harm to the consumers and taxpayers. Municipal ownership is bound to come at Denver, as elsewhere. It is highly desirable that no more money be wasted there on competitive water-works and that the legitimate investment of the company in its works should be returned to it.

By so doing one more chapter in the struggle for municipal ownership of water-works in the United States will be brought to as happy a close as is possible.

LETTERS TO THE EDITOR

Retempered Mortar as a Bond between New and Old Concrete

Sir—I was much interested in the statement of your correspondent in a recent issue (Apr. 3, 1913, p. 689) with regard to the retempering of mortar to make a plaster coating stick to an old wall, as I had been told the same thing several years ago by J. Waldo Smith, Chief Engineer of the Board of Water Supply of New York City. He had inquired of an old mason who had excellent success in making a very tight job how he did it and was told the secret lay in retempering the mortar. I am also told by Mr. Currie, a foreman of tile laying for the firm of Waldo Brothers, of Boston, that it is almost a universal practice in the British Islands to use a certain portion of retempered mortar in the setting of tile, especially where the tile is to be placed on a wall.

The term "retempering," however, is hardly a fair description, for the mortar is mixed and allowed to set for at least 24 hours. It is then hard enough easily to support the weight of a man. This partially set mortar is cut down with a shovel so that it becomes coarse sand and is then mixed with about an equal proportion of fresh sand and cement. It is stated by Mr. Currie that mortar of this kind has a much better body than green mortar and that when placed against the wall, it holds its shape and will not form cracks, whereas if green mortar is used, the tendency is for the cement to run to the bottom behind the tile and a general sagging takes place, which is absent in connection with the retempered mortar.

This may in part explain the reputed advantage of the retempered mortar. The practice of tile setters in this country has been to add a small proportion of lime to the green mortar, a trick well known to brick layers.

In a comparative test of the green cement with an addition of lime and of [illegible] mortar consisting in part of pulverized, partially set mortar, the adhesion of tile set for an interval of five days seemed to be about equal. In each case the mortar had taken a firm hold to the concrete surface, but the green mortar had sagged enough to form two crescent-shaped cracks, of which none were evident in the mortar made up partly of old stock.

Mr. Currie tells me that mortar mixed with lime sets with much less ultimate strength than that mixed with the partially set mortar. This he has tested by cutting out tile on repair jobs where it has been in place for four or five years and in every case, the mortar mixed with old stock was the stronger.

Everyone who has tried to plaster a thin cement mortar coat on old concrete knows of the tendency of the mortar to break down and bulge, due to its fluidity, and if the use of some old stock in the mortar will reduce its quakiness and tendency to run, it can readily be seen that a better job will result.

ALBERT A. NORTHROP.

Keokuk, Iowa, July 5, 1913.

The Guarantee of Estimates by Engineers

Sir—I noticed in your issue of June 5, an editorial article on the guarantee of estimates by engineers.

Your editorial position was correct in the statement that if an engineer is going to absolutely guarantee the cost of the work with a bond in sufficient amount to protect the owner, his position is that of a contractor rather than an engineer and he should take the contract as a contractor, straight, and make what he can on it; because it is not worth while for an engineer to take the gambling risk involved in the execution of work unless he has the opportunity to make the profit which should accompany such a risk.

On the other hand, as a correspondent in your issue of July 10 shows, the estimates made by engineers, particularly when these estimates are made in the course of a competition, are not always to be relied upon. How then shall the City Council and the County Board determine which engineer is playing fair with them and which is not? How are they to know whether the engineer who estimates that a half million dollar structure can be built for $250,000 in order to make a good showing in competition, is treating them in an ethical manner or not? It would seem to the writer that ordinary principles of fair dealing would demand of the engineer making an estimate, particularly when this estimate is given in competition, that he should be willing to give some assurance that the estimate was reasonable and given in good faith. If this is not in accordance with good ethical principles, then it would seem that our ideas of ethical principles need revision since it encourages correct dealing on the part of the irresponsible engineer.

Too often the engineer is held up as a model, a man who is more reasonable in his estimates than the first class and responsible contractor who is ready to take a job and execute it in a first-class manner on the basis of his estimate. It is true that it requires intimate familiarity with work and a broad experience with it to estimate fairly its cost; and it would seem that the engineer who is competing with his fellow engineers should be willing to give a reasonable assurance that his estimates are reasonably close by at least offering to make up complete plans and specifications of his project and to secure bids on them which will be reasonably near his estimate, or else permit the owner to cancel the contract for his services without charge for the work done.

This, I believe, is a fair, ethical way for the engineer to treat the question of guarantee, particularly where his services are engaged as result of competition.

As a commercial proposition, aside from the question of ethics entirely, where the engineer has a patented construction which he is promoting, he ought at least to be willing to guarantee its strength and efficiency just the same as any other responsible business man; and the writer can see no valid objection that could be urged against such a procedure which he has occasionally followed in the course of his business.

If it became a matter of principle among engineers to give the modest guarantee which I have outlined regarding their estimate of the cost of work in competition with their fellow engineers, the profession at large would be benefited since the public would treat the engineers' estimate with a higher degree of respect than they do at present. In competition for public work, particularly for bridges, the engineer possessing the most vivid imagination who can hypnotize himself, together with a Board or Committee, into the belief that a half-million dollar job can be executed for from $300,000 to $350,000, is more liable to secure the contract for engineering services than the man who frankly tells the Board exactly what the work can be reasonably executed for. The modest guarantee I have suggested would make this class of hot-air estimates unprofitable and the public at large would have more respect for the engineering profession and their competency in making estimates. Unreliable estimates, such as I have referred to are far too commonly made by some of the best known members of the engineering profession when competing for public work; and this sharp practice has led to the treatment of the estimate of cost by the professional engineer as more or less in the nature of a joke, to the detriment of the capable and conservative men of the profession.

C. A. P. TURNER.

Phoenix Bldg., Minneapolis, July 12, 1913.

Unit Stresses for Timber

Sir—After a careful reading of your editorial in the issue of June 26, entitled, "What Is a Proper Safe Unit Stress for Timber?" it is apparent that you are laboring under a misapprehension as to some of the conditions surrounding the production and use of yellow pine.

Quoting your editorial:

> One remark which may probably be made in connection with this is that if every structure could be planned and supervised by a competent engineer, then lower factors of safety might reasonably be used on many timber structures. A building law, however, must be made to fit the worst timber which passes the average commercial inspection. As is well known, much of the timber accepted in the present-day market is full of knots and other defects which reduce the maximum strength of many pieces considerably below the average strength which prevailed in the sound and clear timber which was ordinarily obtainable in the market 25 years ago. . . . . There is not one case in ten thousand where those whose lives are at risk in a city building have or can have any knowledge of the strength of the structures in which they live and work.

The paper to which you refer was a discussion concerning certain features of building laws; and in cities that have building laws it is generally the case that plans for all kinds of buildings are submitted to the building department before a permit is granted and that these plans are checked by engineers provided for that purpose, hence the plans and specifications for timber constructed buildings are subjected to the same scrutiny that is given to those for steel-frame or reinforced-concrete buildings.

It is reasonable to presume that equal engineering or architectural ability is employed in the design and construction of timber constructed warehouses, factories and commercial buildings as is employed in the design of steel-frame or reinforced-concrete buildings of the same class—surely ignorance is not confined exclusively to the designing of timber construction. In concrete construction the factors of safety in use today range from 2 to 3½ and the completed structure is required by building ordinances to be "tested" to make sure that they are properly built, and the same "test" could be applied to wood constructed buildings if desired. The defects in timbers are easily observed, their value understood and the proper grading is provided for by the standard grading rules of the Yellow Pine Manufacturers' Association according to which 95% of the yellow pine is graded at the mills in the states of Missouri, Arkansas, Texas, Louisiana, Mississippi, Alabama, Georgia and Florida. It simply involves the same intelligence and precaution to procure a first-class quality of yellow pine that is required to procure a steel or cement of desired quality. Cement and steel are accepted when they comply with specification requirements and there is no reason why the quality of acceptable timber should not be regulated in like manner.

There is as much sense in enacting a law restricting public school education to the capabilities of the intel-

lectually sub-normal, for they undoubtedly exist, as there is in making a law "to fit the worst timber which passes the average commercial inspection"; such laws are distinctly retrogressive and opposed to public welfare, rather give us laws based on a higher principle and enforce them.

It is improbable that there is a Providential intervention that oversees the designing and construction of buildings of materials other than wood and safeguards such operations.

Referring to the "sound and clear timber" of twenty-five years ago, I will say that lumber is "clear" when there is an absence of knots and knots are caused by the growth of limbs or branches.

I have distinct recollection of handling lumber twenty-five years ago and "clear" lumber was restricted to boards used for finishing purposes and structural timbers had the same defects as are today found in timbers of the same grade.

Virgin standing timber is of the same quality today as it was twenty-five years ago and there is 384.4 billion feet of virgin standing southern yellow pine today (see The Lumber Industry, Part I, Standing Timber, Department of Commerce and Labor, Bureau of Corporations, Jan. 20, 1913), and is being cut, manufactured, graded and sold and is in every way equal to that produced twenty-five years ago. By means of intelligent specifications and purchasing it is procured.

ARTHUR T. NORTH.
Consulting Engineer, Yellow Pine Manufacturers' Association.
7056 Eggleston Ave., Chicago, July 9, 1913.

Sir—The writer was much interested in your editorial of June 26, entitled, "What Is a Proper Safe Unit Stress for Timber"? He has not had the opportunity to read the original paper by Mr. North, of which your editorial gives a brief summary. Some of the points which are noted below may have been brought out by Mr. North, but as your summary will probably be read by many who will not have access to the original, the following comments are offered:

(1) Before comparing the stresses allowed by different building ordinances, these stresses must be reduced to the same basis of loads for which the structure is to be designed. It is hardly necessary to mention that the building laws of the different American cities vary considerably in the live-loads prescribed for floors and roofs.

(2) The statement that "if an extreme fiber stress of 1800 lb. per sq.in. is safe in the cities last named, then the cities first named are required by law to use 50% more material in the construction of wooden buildings than is necessary for safety," is hardly correct.

The consideration of allowable deflection is, in many cases, as important as the fiber stresses induced by bending, in determining sections. The same may be said of the consideration of horizontal shear and of bearing across and against the ends of the fibers of the timber. Even assuming that all the working stresses to which the different members of a structure are subjected vary in two different building laws in the same proportion as the fiber stress due to bending, an incorrect assumption, it may still be said that the amount of timber in such a structure will not vary directly with the stresses. The size of roof sheathing, wall sheathing, studding, flooring, and often the roof joists, are determined by other considerations than stress, so that it would be hard to say just what effect on the total amount of lumber in a building a variation in the stress used would produce.

Again, the quantity of the material depends greatly upon the judgment of the designer. No two engineers, given the same building to design, using the same loads, and the same stresses, would produce structures having equal amounts of timber. One engineer might thoroughly brace his building against the wind, the other might consider such bracing superfluous.

(3) It has been the experience of the writer, in checking structural plans, both of timber and steel framing, that the designer in many cases was deceiving himself as to the amount of stresses existing in his structure. Many designers fail to realize the amount of dead-load to be considered. This is especially true with the architect who is generally too optimistic in judging the weight of his materials, especially partitions and fireproofing.

Again, a designer unfamiliar with merchantable timber forgets that his floor joists which he computes as 2x12 in. may be only 1⅝x11⅝ in., due to sizing, possible surfacing, and the fact that any joist over 1½ in. thick sells as a 2-in. joist.

(4) It is worth noting that safe stresses for yellow-pine timber, as recommended by the Committee on Wooden Bridges and Trestles in the 1909 convention of the American Railway Engineering and Maintenance-of-Way Association, are 50% higher than those recommended by the same association in 1895. The working stress for yellow pine in bending, as recommended for buildings by this association, is 1800 lb. per sq.in. At the same time, the specifications for merchantable timber have become less strict in the last few years.

While this report is probably the best authority on timber in the United States, the writer believes that the elastic limit of the timber should be given in all cases. While the yield point of timber is not as marked as that of steel, it is yet a definite quantity, and all safety factors should be based upon it and not upon the ultimate strength. The influence of time on the strength of timber is of more importance than in the case of steel, and a load of less than one-half the ultimate strength will, if continued long enough, sometimes induce failure. Hence, Mr. North's factor of safety of six would better be expressed as a factor of safety of three or less, on the elastic limit. This factor of safety would be true for occasional loads only, and not for continued loading to the full capacity of the beam. It is thus seen that the nature of the loading must be considered along with the allowable stress to be used, and that a general hard and fast rule cannot be made that will be safe at all times and yet be economical.

In conclusion the writer would say that he believes that 1800 lb. per sq.in. for yellow pine in bending is the limit for safety in buildings for dead- and live-loads; but this stress should only be used by a competent engineer experienced in building construction, familiar with the conditions to which the structure may be subjected, and only in a case where it is certain that he will have the supervision of the erection. For building ordinances he believes that a value of 1200 lb. per sq.in. should not be exceeded. The vital point of the whole question is well expressed in your comment that "if every structure

could be planned and supervised by a competent engineer, then lower factors of safety might reasonably be used on many timber structures."

H. D. DEWELL.
Chief Structural Engineer
Panama-Pacific International Exposition, Division of Works.
San Francisco, Calif., July 8, 1913.

Replacing Sills of Chanoine Wicket Dams

Sir—The following information, derived from the "Annales des Ponts et Chaussées" for 1910, Part 3, came to my notice recently when investigating another subject. I do not remember having seen it reproduced in any American or English engineering journal, and it will doubtless interest those who are engaged on the design and construction of Chanoine wicket and other types of movable dams.

The river Seine above Paris is canalized by a series of movable dams with passes of Chanoine wickets, originally constructed between 1860 and 1870, and raised about 20 years later so as to give an additional channel depth (6½ ft. total). The modifications resulted in a maximum depth on the pass sills of about 11 ft. and on the weir sills of 7.8 ft. The lifts varied from 4.4 to 6.8 ft. No changes were made, however, in the substructure, and the original wooden sills were continued in use. (See cross-section, Fig. 1.) About 1905 an examination was made of their condition, and it was found that repairs or replacement would soon become necessary. Not only were the exposed surfaces of the woodwork badly worn by the flow of water, the passage of sand and gravel, and similar causes, but in many cases the shocks from the wickets striking the sill during maneuvers had broken some of the anchor bolts, and in practically all cases had caused an elongation of the holes where the bolts passed through the wood, and the rush of water had enlarged them still further. Many of the bolts fastening the hinge-boxes of the horses had become loosened from the same cause, resulting in a "wobbly" condition of the wickets, and in a few cases the wickets had broken loose during maneuvers.

The sills were of oak, and it should be noted that they were embedded in the masonry for about one-half their height. They were, therefore, in an unusually favorable position for counteracting the effects of shocks.

In the original construction the sills had been made in pieces of two-wicket lengths, or 8½ ft. each, and in order to avoid hindrance to navigation it was decided to replace one length at a time and to set the new pieces under water. It was decided also to make the new sills of reinforced concrete, which, while little if any more expensive than timber, would be comparatively permanent, as well as better able to resist the severe usage which had injured the wooden sills. For reasons of economy the old hinge castings were used again, and holes were left in the concrete so that attachment could be made to the existing anchor bolts. The new sills were made of 1 part of portland cement, 2 parts of sand and 4 parts of gravel, reinforced as shown by Fig. 2. They were cast in molds on shore in lengths of 8½ ft., the hinge castings and their bolts, and the steel-protection plates for the exposed surfaces of the sill being cast in with the concrete. The plates were provided with projections welded to the under sides, anchoring them to the mass. The reinforcement consisted of eight longitudinal bars varying from ½ to ¾ in. in diameter, bent around the castings where necessary. These bars were tied together every 4 in. by wires of $\frac{1}{10}$ in. in diameter.

When replacing a section of sill, a temporary screen or coffer-dam was placed upstream, of a length sufficient to cover two wickets; the wickets were lifted out with the aid of a diver, and the section of the old sill was unbolted and removed. In some cases the last named operation was attended with difficulty, as the sections had been cemented in place, and holes had to be cut under water in order to obtain a sufficient hold for lifting them. After the removal the recess was cleaned out; the new concrete sill was lowered in place; the cement mortar was poured around the anchor bolts, and the nuts were screwed down and covered with mortar.

The cost of making the sills is given as about $4.30

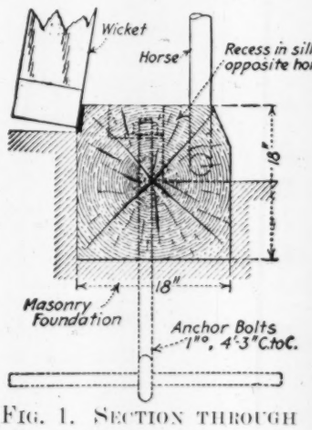


FIG. 1. SECTION THROUGH OLD WOODEN SILL

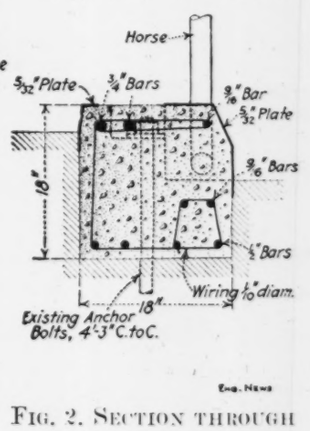


FIG. 2. SECTION THROUGH NEW CONCRETE SILL

per ft. run, and the cost of placing, including diver and coffer-dam and all other expenses, as about $11.10 per ft. run, making a total of $15.40. This was about equal to the cost of using a wooden sill and less than half the cost of a metal sill.

The work of these repairs is in conformity with the best modern design, which endeavors to avoid using any kind of perishable material under water unless it can be replaced easily and without interruption to navigation. It may be noted in this connection that on certain of the recent locks on the Moldau River (Bohemia), wooden miter sills have been omitted, and the lock gates made to close directly against masonry sills. Similarly on the locks of the New York State Barge Canal no wooden sills were employed for supporting the feet of the needles for the temporary coffer-dams, and they rest when in use directly against the masonry of the cross walls.

D. W.

New York City, July 1, 1913.

NOTES AND QUERIES

In the table accompanying the abstract of Professor Hatt's paper on re-rolled rails for concrete reinforcement ("Engineering News," July 10, 1913, p. 75), the side-head "⅜-in. and under" should read "¾-in. and under," in both places where it occurs.

The Third International Road Congress at London

By E. L. Corthell

The International Road Congress has been eminently successful. The Governments of 39 countries are permanent members of the Congress and sent official delegates and there was a large number of private members besides the "corporation" membership consisting of Road and Automobile Associations, Chambers of Commerce, Municipalities, etc.

As an illustration of the interest taken by some countries in the Road Congress, there are nearly 200 delegates and members from Italy, which is preparing to go into road work on an extensive scale and the general and provincial governments as well as municipalities have sent their road engineers here to learn all they possibly can about road construction and maintenance. Large delegations are here from Russia, Germany, Holland, France and other countries. The attendance from the United States is small.

That there is comparatively little interest in this Congress in the United States is in a measure due to the lack of interest in it by the Government at Washington. The late Secretary of Agriculture resisted all appeals to have the United States participate.

The London Congress, while under the general Permanent Commission, composed of delegates of all countries interested in the work, has been organized by the very efficient British Road Board of which Sir George S. Gibb is the director and Mr. W. Rees Jeffreys, general secretary. A general organizing council was selected, composed of Associations of Municipal Corporations, Institutions of Engineers, Chambers of Commerce, Tramway and Automobile Associations. Over 2500 delegates and members were in attendance from nearly all civilized countries. The meetings were well attended; four days of hard work have led to a consensus of opinion and the adoption of conclusions on several important questions.

The general method of preparing for the Congress and conducting its sessions has been similar to that of the Congress of 1910 held at Brussels, of which Engineering News gave a somewhat lengthy report. Questions which were suggested at the latter Congress and those which the Permanent Commission have thought it desirable to discuss and act upon were sent out nearly two years ago to all the countries and to all the permanent members of the Association. Writers on the subjects were selected, reports were received, printed and distributed to the entire membership, a considerable time in advance of the meeting here. All the papers, 123 in number, were placed in the hands of general reporters, or reviewers, selected from the country where the Congress was held, in this case Great Britain, who gave a synopsis of the papers, and following this, a series of conclusions which embodied in general the views of the writers of the papers together with the views of the general reporter. The object was to present to the Congress in a succinct form upon its assembling here general views and resolutions for the Congress to discuss and adopt or amend and reconstruct after full discussion.

The reports of the general reporters were printed in pamphlet form, bound together in a small, convenient volume and were sent out in advance of the Congress so that the delegates and members had a chance to read and study the volume before they came to London. The suggested "Conclusions" were the only matter before the Congress and no time was wasted in reading papers or even the reports of the general reporters; in other words it was an international business congress with no unnecessary oratory to consume valuable time. A brief outline of the work done in the manner above indicated will show how important are the results and how practical the methods of attaining them.

The Congress was divided into two main sections, and these again into two sub-sections each. The nine main questions around which the papers were grouped and with which the conclusions correspond will be taken up in their consecutive order.*

I. Planning New Streets and Roads—The general reporter was Professor Adshead, Department of Civic Design, University of Liverpool, and there were 17 papers submitted. Upon the suggested Conclusions there was an active discussion by delegates and members from Germany, United States, England, Roumania, Belgium, Italy and Austria. The decisions of the Section after discussion and amendment of the proposed Conclusions embraced questions of general "layout" and "bypass" roads, grades, radii, location of tramway tracks, separation of traffic and the question of general supervision by a central authority.

II. Construction and Maintenance of Roads Outside and in Towns—General Reporter, Dr. P. C. Cowan, president of the Institution of Civil Engineers of Ireland. Special subject, "Types of Surfacing on Bridges, Viaducts, Etc." Eight papers. There were participants in the discussion from Russia, England, Hungary and Roumania. The resolutions as adopted covered the entire question for bridges in city and country and included eleven special features and several kinds of surface—plank, not to be adopted, ordinary and tar-bound macadam, wood block, asphalt, stone paving laid on concrete and bound with cement or pitch and the special subject of surfacing on movable bridges.

III. Construction of Macadamized Roads Bound with Tarry, Bituminous and Asphalt Materials—General Reporter, J. Walker Smith, chief engineer, Local Government Board of England. Twenty-five papers. As this general subject was one of the most important of the Congress and one where there is the greatest diversity of opinion, and especially important on account of the new method of traffic by motors, the detailed subjects will be of interest as will be seen by the following copy of the requirements put upon the writers:

> It was desired that Reports sent in upon this general question should include specific comment upon all or some of the following sub-heads, viz.:
>
> 1. Foundations and drainage.
> 2. Sizes and shapes of broken stone for bituminous-bound surface crust.
> 3. Use of partially-worn materials in bituminous-bound surface crust.
> 4. Thickness and composition of the strength crust and of the super or wearing crust under different conditions.
> 5. Life of surface crust under different conditions of traffic, weather, subsoil, etc.
> 6. Relative importance of patching, repairs, and periodical renewals of surface crust.
> 7. Extent of wear permissible before renewal of surface coating.
> 8. Measurement of wear and appliances used for this purpose.
> 9. Various method of bituminous treatment.
>
> (a) Mixing methods:

*The Conclusions suggested to the Congress, with the changes made by the Congress indicated, were printed in full in "The Surveyor and Municipal and County Engineer" (London, England) for July 4, 1913.—Ed.

(1) in factories;
(2) on the spot.
(b) Grouting methods.
(c) Carpeting methods.
(d) Other methods.
10. Relative advantages and use of tar, tarry compounds, asphalt, bitumen, and other materials.
11. Tests and chemical analysis of tarry, bituminous and asphaltic compounds.
12. Climatic effects causing slipperiness of the roadway—Remedies.
13. Effect on public health, fish, life, or vegetation.
14. Specification of the methods of construction.
15. Cost data.
16. Cleansing and watering.

The questions were discussed by delegates and members from France, Russia, Austria, Belgium, Roumania, Greece, Great Britain, Italy, India, United States and Australia. As the subject is of great interest and requires further investigation and tests, the following "General Conclusions" were agreed to:

By the use of bituminous, including tarry or asphaltic, binders we may obtain a number of different forms of road crust, which may be employed with advantage, according to the various conditions of the road as regards traffic, locality, and climate.

The exact value and duration of life of these various road crusts, taking into account traffic, climatic conditions, and the methods of construction, remain to be determined.

For this purpose it is advisable to draw up a uniform system of tests, measurements and records, under the following headings:

1. Physical and local conditions. (Plans, cross-section, slopes, camber foundations, subsoil).
2. Materials employed, petrological analysis, dimensions, composition of the binding agent.
2a. Method of construction, date of construction.
3. Census of traffic on the section under review.
4. Climatic conditions affecting the road.
5. Periodical measurement of wear.
6. Periodical examination of the state of the road crust.
7. Actual cost of the road crust:
(a) as regards cost of construction.
(b) as regards maintenance cost.

The standard form in which the information is to be furnished will be drawn up by the Permanent Commission.

"Particular Conclusions" were also agreed to on foundation and drainage, dimension and shape of metalling (stone) employment of partially used materials, relative importance of patching, permissible wear, various means of employing tarry, bituminous and asphaltic materials, tests and chemical analyses, climatic effects, effects on public health, cleansing, watering.

The following proposition was adopted in closing the debate:

An international technical Committee should be appointed by the Permanent International Commission in order to study a standard method of obtaining information and data upon materials, physical conditions, local conditions, methods of construction, terminology and other points concerning macadam bound with tarry, bituminous or asphaltic binders.

The report of the Committee should, after examination by the Permanent Commission, be presented to a next Congress.

IV. Wood Paving—General Reporter, Mr. Percy Boulnois, Member of the Institution of Civil Engineers, London. Papers were presented from Hungary, Russia, United States and France, and a joint report from four engineers of Great Britain. The subject was discussed by members from Great Britain, Germany, United States and France. The conclusions agreed to may be condensed as follows:

Wood paving permissible where grades will allow it and traffic is great but not too heavy, but a concrete foundation is always necessary. Great care is required in selection of the proper timber, all soft wood blocks to be impregnated with some well-proven preservative before laying. Further laboratory experiments necessary. Close joints to prevent the entrance of water. Hard woods give varying results; not desirable to employ it for streets with intense traffic in large cities unless the joints can be prevented from destruction with resulting destruction to the concrete below. From sanitary point of view no objection to wood paving if kept clean and the wood impregnated with preservatives and the joints and surface waterproof. Sanding or graveling necessary to prevent slipperiness.

V. Methods of Lighting—General Reporter, Col. H. C. L. Holden, Royal Automobile Club, London. Papers were presented from Austria, Belgium, France, Germany, Great Britain, Russia and the United States, nine in all, and the subject was discussed by members from France and Great Britain. The following is a summary of the conclusion adopted on this question:

Highways for purposes of general determination of methods of lighting may be conveniently divided into three classes as follows:

(1) Main streets in cities, towns or other urban areas in which the traffic after dark is considerable in volume.
(2) Important suburban roads in the vicinity of large towns.
(3) Rural roads and open country.

Where highways are lighted the lighting should be uniform and free from glare. Importance of all vehicles being lighted on rural roads whether moving or standing; even herds of cattle should be indicated by lights at night. Details of lights for motor cars. All gates, especially at crossings of railways to be painted white and illuminated with fixed lights. All danger and signal posts to be painted white, also direction posts, mile stones, bridge abutments, railways and so forth and all danger signals should have a uniform color. It is desirable that each Government should do away as soon as posible with colored lights on automobiles.

VI. Observations Since 1908 (Date of the First Road Congress) as to Various Causes of Wear and Deterioration of Roadways—General Reporter, William Gibson Thompson, editor of "The Surveyor," London. Papers were presented from Austria, United States, France, Great Britain and Holland, and the subject was discussed by members from Great Britain and Italy. Generally the conclusions were:

That weather conditions are the most important causes of deterioration to be minimized by waterproofing the surface and good drainage for the foundations. Water-bound macadam is seriously damaged by heavy motor vehicles or high-speed motor cars. The damage is effected by the balancing of the vehicle, ratio between propelling power and adhesive weight, weight of unsprung portion of the motor, the progressivity of action of the brakes, the system of springing, the type of tire employed, the diameter of the wheels, the width of the rims, variations of speed and adherence and other factors. Damaging effect can be minimized by wheels of large diameter adapted to the weight of the axle load; rubber tires, and suitable springs. Light motor car traffic does not cause serious or exceptional wear on properly made macadam roads treated or bound with tarry, bituminous or asphaltic materials, except on sharp curves. Horse-drawn vehicles to be properly tired and wheels of proper size, and horses to be properly shod. Mud from fields in agricultural districts should not be brought out on to the roads by the wheels of carts. It is desirable to collect more information compiled on carefully devised scientific methods standardized as far as possible for purposes of comparison and the International Permanent Commission is charged with the preparation of a programme of observations, studies the experiments.

VII. Regulations for Fast and Slow Traffic on Roads—General Reporter, Lord Montagu, of Beaulieu, of London. Papers were presented from the United States, Austria, France, Germany, Great Britain and Holland. The subject was discussed by members from France, Great Britain, Belgium, Germany, Austria, Italy and Holland. The conclusions are important:

All regulations to be based on allowing the practicable speed consistent with public safety, general convenience, and normal wear of the road. Regulations to be as few and as simple as possible. All large cities to have a "traffic authority" to deal with all these questions with the necessary powers and to include adequate police supervision. Drivers to be carefully and systematically trained and children to be specially taught how to provide against the dangers of the road. No obstructions like lamp-posts and tramway standards to be placed in the center of the road except the necessary refuges for pedestrians crossing. No obstructions

of a public highway by vehicles standing unnecessarily or moving at an obstructive speed.

VIII. AUTHORITIES IN CHARGE OF CONSTRUCTION AND MAINTENANCE OF ROADS; FUNCTIONS OF CENTRAL AND LOCAL AUTHORITIES—General Reporter, W. Rees Jeffreys, general secretary and secretary to the Road Board of Great Britain. This subject was exhaustively treated by the General Reporter after receiving nine papers from Belgium, Bulgaria, Canada, France, Germany, Hungary, Italy, Great Britain, United States and Russia. It was discussed by members from Great Britain, France and Germany. After an important discussion at the final meeting of the Congress where the two sections were united the two following features were adopted:

The system of road administration in any country must be in harmony with the general system of government prevailing in that country, and the political genius of its people. It is impossible, therefore, to lay down any general rule of universal application as to the extent to which the road organization of any country should be centralized or decentralized.

A principle that can be laid down as of universal application is, that the unit of highway administration shall be sufficiently large and command sufficient resources to employ and adequately remunerate a competent staff.

IX. FINANCE OF THE CONSTRUCTION AND UPKEEP OF ROADS; PROVISION OF REVENUES—General Reporter, G. Montagu Harris, Barrister-at-law, secretary of the County Councils Association, England and Wales. There was discussion by members from Germany, France and Great Britain. In substance the conclusions were:

Expenditure upon main roads of communication between important places and upon roads used principally for long distance traffic should be mainly paid out of national revenues. All tolls should be abolished on public roads but it is equitable that vehicles which, on account of their weight or weight combined with speed or any other exceptional circumstances connected with either the vehicle or use of the road, cause special damage to roads beyond the wear and tear of the ordinary traffic of any district, should be subject to special taxation the proceeds of which should be earmarked for expenditure on roads.

It is sound finance to borrow money for new road construction and for periodic surfacing providing "that the loan period in the case of loans for renewals is kept well within the life of the surface coating."

The papers, general reports, discussions and the conclusions finally adopted on these nine important subjects will prove to be a literary product of great usefulness to all engineers and all authorities, national, state, provincial, county, town and municipal, and it goes without saying that all these parties will greatly benefit by putting themselves in the position to receive the literature of this Permanent Road Congress, the small cost of membership placing its advantages within reach of all interested.

Some of the many functions and excursions, those during the Congress and those after it, may be outlined as follows: A Reception at the Road Exhibition at the Royal Horticultural Hall; Reception and Conversazione at the Guildhall, by the Lord Mayor; Reception by the President of the Institution of Civil Engineers, at the Royal Albert Hall, on the occasion of its annual Conversazoine. Inspection of roads in Essex, 50 in the party, and a tour of inspection of roads in Berkshire, 100 in the party, over tar-macadam roads for about fifty miles, with an examination of the work going on at some points. Reception by the Royal Automobile Club. All of the above, and more, was on Thursday, June 26.

Garden Party by the President, Council and Members of the Surveyors' Institution at the Royal Botanical Gardens. Excursion to the National Physical Laboratory, including the Road Board Laboratory for testing road materials at Bushey and to Hampton Court Palace, 180 in the party. Tour of inspection of roads in Surrey and Sussex, 50 in the party. The above on Saturday, June 28, followed by the final banquet and reception, at the Hotel Cecil, which closed all the work and functions of the week.

There were previous excursions of the Congress to examine roads in South London and Sidcup with full data for the participants. On the Berkshire excursion each member was presented with a pamphlet with a map of the county showing the main roads and particularly the roads traveled over. All roads passed over were fully described in the pamphlet with materials stated and method of construction.

After the Congress the excursions for examining the roads of England, Wales and Scotland were as follows: To North Wales from Chester by automobile and char-a-banc returning to London by rail from Shrewsbury, 50 in the party, five days; Liverpool and Sheffield, 50 in the party, six days; to Glasgow, 50 in the party, six days; to Sheffield, Leicester, etc., 100 in the party, four days; London to Folkestone, 50 in the party, two days.

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The Third International Road Congress

BY NELSON P. LEWIS*

The Third International Road Congress met in London on June 23, and continued for one week. More than thirty different countries were officially represented, the United States being the only important nation which did not send delegates, and which has not become a member of the Permanent Association of International Road Congresses. At the first Congress held in Paris in 1908, and at the second Congress held in Brussels in 1910, the United States government, though not a member of the Permanent Association, sent official representatives who were treated with every courtesy, although this country was and still remains without representation in the management of the Permanent Association.

In the Deficiency Appropriation Bill, which became a law on Mar. 4 last, Congress inserted a provision forbidding the Executive to either extend or accept any invitation to any International Congress, convention or other meeting without the express authority of Congress. The American delegates who attended the International Road Congress this year without expense to the federal government were consequently placed in a very awkward position, and it was impossible to make the representatives of other governments understand whether the refusal of the United States to send officially accredited delegates was due to provincialism or to a deliberate lack of courtesy to other countries, and especially to the British Organizing Committee of which Sir George Gibb, of the Road Board, was Chairman.

The United States, however, was well represented in the papers which were presented for consideration, having contributed to eight of the nine "Questions" and nine of the ten "Communications."

The registered attendance at the Congress was well over 2000, and the only feasible method of presenting and discussing the various subjects appeared to be that followed in the two preceding Congresses. One paper on each of the "Questions" and "Communications" was accepted from each country, although several collaborators were permitted to join in the presentation of a single paper. A general reporter was designated for each "Question," and by him the various papers upon the "Question" were reviewed, and his conclusions were placed in the form of resolutions for consideration at the sectional meetings where they were discussed and

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amended. The resolutions so amended by the sections were presented at the final meeting of the Congress, and after further amendment in some cases were adopted as the expression of the views of the Congress. The "Communications" were not presented for discussion, although many of them contained valuable material.

The ten subjects of "Communications" were:

1. Improvements adopted since the second Congress in machinery used in construction and maintenance of macadamized roads.
2. Tests of materials used in the construction of macadamized roads.
3. Construction of water-bound macadamized roads.
4. Technical and economic study of the comparative advantages of different types of roads.
5. Various types of stone paving in use.
6. Direction and distance sign-posts.
7. Development since the second Congress in self-propelled public-service vehicles.
8. Qualifications of Engineers and Surveyors in charge of the construction and maintenance of roads.
9. Statistics of costs of construction and maintenance.
10. Terminology adopted or to be adopted in each country relating to road construction and maintenance.

Space does not permit even a most general review of the various reports and communications. They were essentially engineering papers, but some of them were too long and presented in a manner too technical for a Congress of this kind. The papers written in other languages were well translated into English; and if the writers who employed the English language were as well translated into the other two official languages of the Congress, they were fortunate.

While the details of the meetings, the excursions and the various functions were exceedingly well arranged and carried out, there was one unfortunate incident which rendered it difficult for those in attendance to intelligently follow the proceedings and discussions at the sectional meetings, viz., the absence of copies of the general reports with reviews of the different papers and the resolutions presented for discussion. These were printed in Paris and appeared to have been mailed to the delegates at their home addresses, but failed to reach them before they left, so that a very small proportion of the delegates had before them anything but the brief abstract contained in the official *Journal* of the Congress which was distributed each morning.

It is probably inevitable that in an International Congress dealing with technical questions, no resolutions can be adopted except those which are expressed in very general terms. A resolution which contains an explicit declaration as to theory or practice is almost sure to be questioned by the representative of one or more countries where the prevailing methods are somewhat different, and the feeling of courtesy which usually prevails at such meetings will prompt those who are quite sure of their ground to defer to the hesitancy of those who have not yet reached a definite judgment. As an illustration of this tendency, reference may be made to the conclusions presented by Mr. Rees Jeffreys, the General Reporter for Question 8 on "Authorities in charge of the construction and maintenance of roads." These resolutions embodied some excellent ideas, but as they appeared to be new to some members of the Congress representing local administrations, they were reduced to two perfectly harmless but somewhat meaningless paragraphs. The resolutions which were finally adopted contained little that is new, and no attempt will be made to review them.

It does not follow that the Third International Road Congress was barren of results to those who attended it. The most conspicuous advantage of meetings of this kind is the opportunity afforded those attending them to observe the practice of other countries and cities, and to see what they have done, and are doing, and how they do it, rather than to bring back with them resolutions briefly expressing the fundamental principles by which they are governed.

Perhaps the feature of the Congress most valuable to American delegates was the opportunity to observe the extraordinary improvement which has taken place in the English roads during the past three years. This is true both of the city streets and the rural highways. The road investigations carried on at the National Physical Laboratory at Bushey Park, clearly indicate that the English highway engineers are not floundering about in the hope that they will be able to stumble upon the right methods of construction and caring for their roads, but that they are approaching a difficult problem in a thoroughly scientific manner. The general appreciation of the national importance of the highway problem was also indicated by the admirable addresses of the Right Hon. David Lloyd George, Chancellor of the Exchequer, and by Sir George Gibb, President of the Congress at the opening session, and that of the Right Hon. John Burns, President of the Local Government Board at the final banquet at the Hotel Cecil.

The next Congress is to be held at Munich, Germany, in 1916. There is good reason to believe that those who were present at London this year would be glad to come to this country fir the next following Congress; but so long as the United States government holds aloof and does not indicate its interest in this world movement in favor of highway improvement, it naturally can extend no invitation to other nations.

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A New East River Tunnel at New York City was holed through on July 17. This is a service tunnel for the Consolidated Gas Co., and extends from Fort Morris in the Borough of the Bronx to Astoria in Queens. It is 21 ft. x 19 ft. 9 in. in section, 4662 ft. long and is to carry for the present two 6-ft. 6-in. gas mains and some minor water mains. Construction has been in progress for nearly three years.

Two Wrecks Occurred within a few hours of each other, on the Michigan Central R.R., near Jackson, Mich., July 13. The first involved a train of empty baggage cars, running from Chicago to Buffalo, that left the rails. The engineer was killed and the fireman severely scalded by the escaping steam from the boiler. A few hours later two passenger trains on the Grand Rapids division were wrecked in a butting collision. The causes of the two accidents are not definitely known.

A Locomotive Headlight Was Mistaken for a Lightning Flash by the conductor of a street car at a crossing in East Cambridge, Ohio, July 13, according to press dispatches. It is reported that the conductor had stopped his car and gone forward to the crossing and signaled the motorman to proceed. The accident occurred during a terrific storm and the oncoming headlight of the westbound Wheeling-Chicago express of the Baltimore & Ohio R.R. was not recognized. The express struck the street car just as the latter reached the middle of the crossing, killing four persons and injuring 16.

Two Mechanical Filtration Plants for the water supply of Cleveland, Ohio, have been recommended by the Filtration Commission of that city. Of these, one of about 150,000,000 gal. capacity should be built, the Commission says, at the Division St. pumping station, and (later on) one of about 80,000,000 gal. capacity at the Kirtland St. pumping station. These two plants would supply double the present consumption. General sketch plans for each of the proposed plants have been submitted by the Commission. The estimated cost of the two plants, and of necessary low-lift pumping machinery, is $2,000,000, of which $1,100,000 and $700,000 are

for the larger and smaller filter plants, respectively, and $200,000 for the pumping plant. The estimated operating expenses, exclusive of interest and other fixed charges, are "from $2 to $4 per 1,000,000 gal., according to the amount of coagulants used." On the basis of a water consumption of 100 gal. per capita the operating cost just given "represents an annual charge of not more than 15c. per person" and the "annual charges would be approximately 12c. per capita additional," for a population of 1,000,000. Questions relating to the kind and amount of coagulants, water softening, etc., are now being studied at the testing plant installed at the Kirtland St. station. It would be unnecessary to build the Kirtland St. plant at once, as after the completion of the West Side tunnel in 1915, it is proposed to supply the whole city for a time from the Division St. station. R. Winthrop Pratt, consulting engineer, Cleveland, and formerly chief engineer of the Ohio State Board of Health, has been appointed consulting engineer to the Cleveland Water Department. Mr. Pratt is secretary of the Filtration Commission and to him we are indebted for a brief preliminary report from which the foregoing information has been taken.

The Offer of a $10,000 Prize for the best automatic stop and speed control device suitable for railway use, made by the New York, New Haven & Hartford Railroad, has brought out offers of inventions by 2816 persons up to July 1, when the time limit expired for submitting designs. Of this total number, 704 have submitted plans of their devices in reply to the circulars sent to each applicant giving the conditions of the competition. The conduct of the competition is in charge of C. H. Morrison, chief signal engineer of the New Haven system. When inaugurating the competition, the New Haven company laid down the requirements formulated by the late Block Signal and Train Control Board of the Interstate Commerce Commission and added eight additional requirements of its own. It is stated that not one of the appliances so far submitted complies directly with condition No. 1, which reads as follows:

> The apparatus should be so constructed that the removal or failure of any essential part would cause the display of a stop signal and the application of the train brakes, and if electric circuits are employed they should be so designed that the occurrence of a break, cross, or ground, or a failure of the source of energy in any of the circuits should cause the display of a stop signal and the application of the train brakes.

It is also stated that most of the devices fail to embody speed control feature and also neglect the requirement that the device should work under snow, ice, sleet, and freezing conditions. Road tests will be made, however, of two of the devices which come the nearest to fulfilling the conditions laid down. One of these is invented by Gene Webb, and is owned by the International Signal Co. The other is the invention of an engineer connected with the Union Switch & Signal Co. Both of these systems make use of an electric current transmitted by a third rail. A section of track will be equipped with each system on the western division between Hartford and Newington. Similar tests of these systems are to be made by the Lackawanna Railroad. The tests will probably continue at least a year, in order to see how the devices endure the extremes of both summer and winter weather.

PERSONALS

Mr. A. D. Williams, of Marlington, W. Va., has been appointed Road Engineer of West Virginia at a salary of $3500 per annum.

Mr. H. J. Osborne has been appointed Superintendent of Motive Power of the South Dakota Central Ry., at Sioux Falls, S. D., succeeding Mr. L. J. Shannon.

Mr. D. C. Douglas, recently Assistant to the Vice-President and General Manager of the Maine Central Ry., at Portland, Maine, has been appointed Assistant to the President.

Mr. Heng T. Hu, a young Chinese graduate of Lehigh University, has received a temporary appointment on the engineering staff of the New Jersey State Board of Assessors.

Mr. Walter F. Wilson, Superintendent of Highways of St. Lawrence Co., N. Y., at Louisville, N. Y., has been appointed Deputy Commissioner of Highways of the State of New York.

Mr. George W. Manning, formerly Assistant Superintendent of railway service, Utah Light & Railway Co., Salt Lake City, Utah, has been promoted to be Superintendent of railway service.

Mr. W. E. Dozier has resigned as City Engineer of Temple, Tex., to become General Superintendent and Chief Engineer of the Temple, Northwestern & Gulf Ry., a new railway about to be constructed.

Mr. E. M. Costin, formerly Division Superintendent of the Cleveland, Cincinnati, Chicago & St. Louis R.R., at Cleveland, Ohio, has been promoted to be Assistant General Superintendent at Indianapolis, Ind.

Mr. F. S. Robbins, formerly General Foreman of the car shops of the Pennsylvania R.R., at Pitcairn, Penn., has been promoted to be Assistant Master Mechanic of the Pittsburgh division, at Pittsburgh, Penn.

Mr. C. B. Daily, Assistant Superintendent of Shops of the Chicago, Rock Island & Pacific Ry., at Silvis, Ill., has been appointed Master Mechanic, at Cedar Rapids, Iowa, succeeding Mr. F. W. Williams, transferred.

Mr. H. L. Lyon, formerly Roadmaster of the Southern Pacific Ry., at McMinnville, Ore., has been appointed Division Engineer of the Second division of the Oregon-Washington R.R. & Navigation Co., at La Grande, Ore.

Mr. R. Winthrop Pratt, M. Am. Soc. C. E., has been appointed Consulting Engineer of the Water Department of Cleveland, Ohio, on the design of a 225,000,000-gal. filtration plant. For the past year Mr. Pratt has had charge of the sewage disposal investigations for the City of Cleveland.

Mr. P. T. Dunlop, Mechanical Superintendent of the Gulf, Colorado & Santa Fé Ry., at Cleburne, Tex., has resigned, effective Aug. 1, to become General Superintendent of Motive Power of the St. Louis & San Francisco R.R., at Springfield, Mo., succeeding Mr. George A. Hancock, resigned.

Mr. Charles Murphy, former General Superintendent of Transportation of the Canadian Pacific Ry., at Montreal, Que., has been appointed General Superintendent of the Manitoba division, at Winnipeg, Man., succeeding Mr. D. C. Coleman, who has been transferred to the Alberta division at Calgary.

Mr. T. W. Norcross, Assoc. M. Am. Soc. C. E., formerly District Engineer, U. S. Forest Service, Denver, Colo., has been promoted to be Assistant Chief Engineer of the Forest Service with headquarters in Washington, D. C. Mr. Norcross is at present engaged in work in connection with power and irrigation projects in the National Forest Reserves.

Mr. H. F. Thomson, Assoc. Am. Inst. E. E., Research Associate in the department of electrical engineering, Massachusetts Institute of Technology, has been promoted to be Secretary of the Research Division in Electrical Engineering and Assistant to the Director of the division. Mr. Thomson graduated from Washington University, St. Louis, Mo., in 1910.

Mr. W. D. Stanton, Assoc. M. Am. Soc. C. E., recently Assistant Engineer, Central Division, Panama Canal, has been promoted to be Assistant Engineer in charge of sluicing operations, Gold Hill, Culebra, succeeding Mr. W. L. Thompson, M. Am. Soc. C. E., resigned to become Chief Engineer of the Mississippi State Levee Board, as noted in our issue of July 10.

Mr. G. W. Boschke, Assistant General Manager of the Oregon-Washington R.R. & Navigation Co., has been appointed Chief Engineer with jurisdiction over the First division south of the Columbia River and the Second, Third, Fourth and Fifth divisions and all construction work adjacent to these lines with the exception of the Spokane Union Terminal, with offices at Portland, Ore.

Mr. J. R. Holman, Assistant General Manager of the Oregon-Washington R.R. & Navigation Co., in charge of construction, has been appointed Chief Engineer with jurisdiction over the First division, north of the Columbia River, and also of the construction of the Spokane Union Terminal and the line westerly to the Summit Boulevard, Spokane, Wash., with offices at Seattle, Wash.

Mr. Alfred Price, formerly General Superintendent of the Canadian Pacific Ry. at Calgary, Alta., has been promoted to be Assistant General Manager at Montreal, Que., succeeding to the duties of Mr. C. Murphy, General Superintendent of Transportation, which position has been abolished. Mr. Price is 52 years old and entered the service of the Canadian Pacific Ry. in 1882 as a telegraph operator.

Mr. J. Q. Van Winkle, recently General Manager of the Cleveland, Cincinnati, Chicago & St. Louis R.R., has been appointed Assistant to the Vice-President, J. J. Bernet. He is succeeded as General Manager by H. A. Worcester, formerly Assistant General Manager. Mr. Van Winkle is 62 years old and has been continuously in railway work since he was 19 years old. He has been General Manager for seven years.

Mr. R. S. Stephens, Chief Engineer of the Missouri Pa-

cific Ry. system, has been temporarily relieved of his duties in order to devote his whole time to duties in connection with the valuation of the railway property under the new Interstate Commerce Law. Mr. C. E. Smith, Bridge Engineer, has been promoted to be Assistant Chief Engineer and Acting Chief Engineer, with offices at St. Louis, Mo., as noted elsewhere in these columns.

Mr. Benjamin F. Bush, President of the Missouri Pacific Ry. system and the Denver & Rio Grande R.R., at St. Louis, Mo., has been elected President of the Western Pacific Ry. also, succeeding Mr. Edward T. Jeffrey, who becomes Chairman of the Board of Directors. Mr. E. L. Brown, Vice-President of the Denver & Rio Grande R.R., at Denver, Colo., has been elected Vice-President of the Western Pacific Ry. also, and will hereafter make his headquarters in San Francisco, Calif.

Mr. Henry J. Horn has resigned as Vice-President in charge of operation of the Boston & Maine R.R. Mr. Horn was, until recently, a Vice-President of the New York, New Haven & Hartford R.R., and was placed in charge of the operating department of the Boston & Maine R.R. in June, 1912, when the managements of the two railways were combined. He was succeeded as Vice-President in charge of operation of the New York, New Haven & Hartford R.R. last January by Mr. A. R. Whaley.

Mr. Philip D. Borden, whose resignation as City Engineer of Fall River, Mass., was noted in our issue of last week, has been appointed a member of a city commission to investigate the nuisances along the Quequechan River and the question of the needs of the city for an increase, purification, protection and control of its public water supply. The commission is authorized to make a complete survey of the watershed and to prepare plans with the approval of the State Board of Health for protecting or purifying the water-supply.

Mr. J. L. Dobbins, Assoc. M. Am. Soc. C. E., has resigned his position with Haviland, Dozier & Tibbetts, Civil Engineers, San Francisco, Calif., to return to China as Assistant Chief Engineer of the Yunnan, Szechuan & Tengyueh Ry., with headquarters at Yunnanfu, China. Mr. Dobbins was for some years Professor of railway engineering at Peiyang University, Tientsin. Until a short time ago he was also Consulting Engineer at Tientsin. Mr. Dobbins left San Francisco on July 16 and will arrive at Yunnanfu, some time in the early part of September.

Mr. J. M. Harris has been appointed Division Engineer of the Western Maryland Ry. in charge of maintenance-of-way of the Maryland division, comprising the lines east P. V. Junction, with office at Hagerstown, Md., succeeding Mr. J. Carmichael, formerly Engineer of maintenance-of-way, who has been assigned to the Chief Engineer's office at Baltimore, Md. The positions of Engineer of Maintenance-of-way have been abolished; Messrs. W. C. Kline and P. Cain have been appointed Division Engineers in charge of maintenance-of-way at Connellsville, Penn., and Elkins, W. Va., respectively.

Mr. Ralph H. Sweetser, Superintendent of the Columbus Iron & Steel Co., of Columbus, Ohio, has been elected President of the Thomas Iron Co., of Easton, Penn. Mr. Sweetser is a native of Portsmouth, N. H., and a graduate of the Massachusetts Institute of Technology, in mining engineering and metallurgy, class of 1892. He has been connected at various times with the Maryland Steel Co., the Everett Furnace, Everett, Penn., the Salem Iron Co., Leetonia, Ohio, the Cranberry Furnace Co., Johnson City, Tenn., and from 1904 to 1907 he was Superintendent of blast furnaces of the Algoma Steel Co., Sault Ste. Marie, Ont. Since 1907 he has been Superintendent of the Columbus Iron & Steel Co.

Mr. R. C. Watkins, formerly Right-of-Way Agent of the Southern Pacific Ry. lines in Louisiana and Texas, has been promoted to be Superintendent of the Houston division of the Galveston, Harrisburg & San Antonio Ry., at San Antonio, Tex. Mr. Watkins graduated from the Agricultural and Mechanical College of Texas in 1895 in civil engineering. His first practical experience in railway work was that of a track laborer; later he was assistant yard foreman. After spending about two years with a railway construction company and with the Edison Electric Co., of New Orleans, La., he was appointed Assistant Engineer of construction and maintenance of the Louisiana Western and Morgan's Louisiana & Texas R.R. & Steamship Co. Later he was Resident Engineer at Houston, Tex.

Messrs. Vitale & Rothery, Forest Engineers, New York City, have just begun what they believe to be one of the largest timber surveys ever undertaken in Canada. The timber tract to be surveyed and appraised embraces the entire watershed of the Lievre River, a concession of 2,000,000 acres belonging to the James McLaren Co., Ltd., of Buckingham, Que. The work will include the determination of the boundaries, an estimate of the timber, a general appraisal, a detailed report on the capacity of reservoirs, the extent of land suitable for agriculture, mineral resources, and all other obtainable information of value in developing the concession. A large part of the tract has never before been surveyed. The work will be under the personal supervision of Mr. Julian A. Rothery, a graduate of the Yale University School of Forestry, for several years in the United States Forest Service and recently Supervisor of the Idaho National Forest. The survey is expected to require the services of 40 technically trained foresters and timber cruisers for 18 months.

Dr. Harold Pender, M. Am. Inst. E. E., Professor of electrical engineering, has been promoted to be Director of Electrical Engineering Research, at the Massachusetts Institute of Technology. Dr. Pender is a native of North Carolina and a graduate of Johns Hopkins University, where he received a Ph. D. degree in 1901. Later he studied in Paris. In 1903 he entered an apprenticeship course at the Westinghouse Eleetric & Manufacturing Co., Pittsburgh, Penn., later being appointed to a position on the engineering staff and given charge of the testing of sheet steel for electrical purposes. In 1904-'05 he was with the electrical engineering department of the New York Central & Hudson River R.R. in work on the design of the New York Terminal distribution system. From 1905 to 1909 he was employed by Mr. Cary T. Hutchinson, F. Am. Inst. E. E., Consulting Electrical Engineer, New York City. Dr. Pender joined the faculty of the Massachusetts Institute of Technology in 1909 as Professor of theoretical and applied electricity. He is the author of "Principles of Electrical Engineering" and Editor of "The American Electrical Engineers' Hand Book."

Mr. E. N. Brown has resigned as President of the National Rys. of Mexico, at Mexico City, due, it is said, to the continual interference of government officials with the operation of the railway. Mr. Brown has been connected with the Mexican National Rys. for many years, and to his engineering, constructive and executive ability is due, in a large measure, their development, and until the recent political disturbances, their financial success. Mr. Brown was born in Barbour County, Ala., during the exciting times of the American Civil War. He entered into railway work at the age of 16 years as a rodman and chainman on the preliminary and location surveys of the Northeastern R.R. of Georgia. Later he was Engineer on construction of the Richmond & Danville R.R. From 1883 to 1887 he was Division Engineer, Resident Engineer and Locating Engineer of the Central of Georgia Ry. In 1887 he went to Mexico where he was Engineer in charge of construction of the Mexican National Ry., from Saltillo to San Luis Potosi. The following year he was made Superintendent of the same division at San Luis Potosi, and in 1889 Division Superintendent at Mexico City. From 1892 to 1902, he was General Superintendent of the Mexican National Ry. In 1902, he was elected Vice-President and General Manager of the company, and from 1904 to 1909 he was President of the reorganized company, the National R.R. of Mexico. Since 1909 he has been President of the consolidated National Rys. of Mexico.

Mr. Charles Edward Smith, Bridge Engineer of the Missouri Pacific Ry., has been promoted to be Assistant Chief Engineer, with the duties temporarily of Chief Engineer. Mr. Smith was born in Somerville, Mass., in 1877. He was educated at the Massachusetts Institute of Technology, where he graduated in 1900, in civil engineering, having previously acquired considerable experience in engineering work. From June to November, 1900, he was Assistant City Engineer of Putnam and Willimantic, Conn. For three years he was a draftsman and bridge designer in the engineering department of the New York, New Haven & Hartford R.R., at New Haven, Conn. After a few months as a structural-steel draftsman with the Bureau of Yards and Docks, Navy Department, Washington, D. C., Mr. Smith returned to railway work with the Lake Shore & Michigan Southern Ry., at Cleveland, Ohio, as Assistant Engineer on bridge erection and inspection work. In 1905, he was a text-book writer for the International Correspondence Schools, Scranton, Penn., and in 1906 and 1907, Technical Editor of the Structural Materials Division, U. S. Geological Survey. Since August, 1907, he has been with the engineering department of the Missouri Pacific Ry., first as Assistant Engineer in the maintenance-of-way department and since September, 1909, as Bridge Engineer. Mr. Smith will be remembered by the readers of "Engineering News" as the author of some interesting papers on Missouri Pacific Ry. bridges